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The "beam method" of analyzing cylindrical shells is briefly explained by JAMES CHINN in his paper "Cylindrical Shell Analysis Simplified by Beam Method."

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Results of experiments carried out to determine "The Flexural and Compressive Strength of Concrete as Affected by the Properties of Coarse Aggregates" are set forth by M. F. KAPLAN.

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L. EARL TABLER, JR., describes unusual construction practices occasioned by limited access to chamber interiors in "Internal Shielding Construction at Shippingport Nuclear Power Plant."

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F. T. MAVIS and J. J. STEWART in "Further Tests of Dynamically Loaded Beams" report on 27 beams tested to destruction by spring-actuated impulse loads applied at third points of an 8-ft simple span. All beams were identical except for amount and grade of reinforcement.

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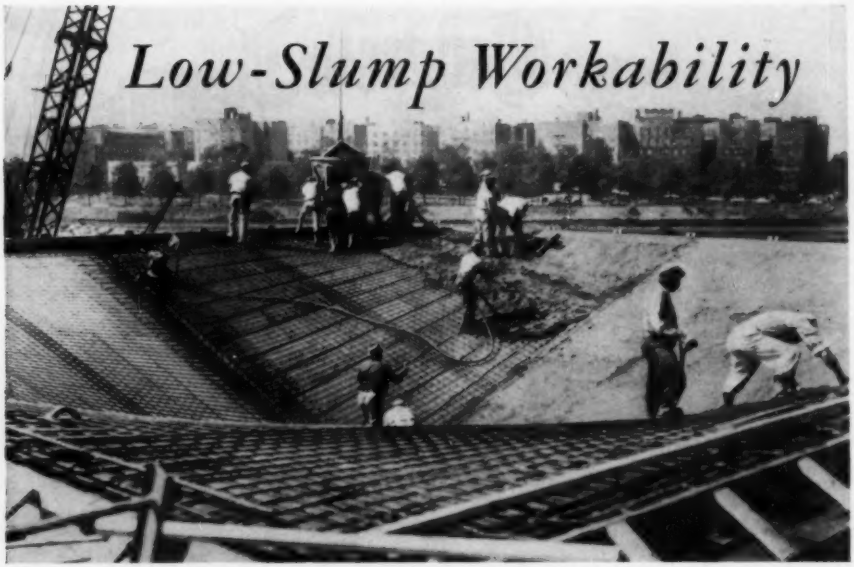

This hyperbolic paraboloid, spanning 60 feet, is one of six for the new library of Hunter College, New York City.

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Title No. 55-64

DOUGLAS McHENRY, director of development, Portland Cement Association, Skokie, Ill., retiring president of ACI, discusses the concrete industry with respect to other fields of scientific endeavor and its own history and future.

DYNAMICS AND STATICS IN CONCRETE INDUSTRY PROGRESS

By DOUGLAS McHENRY

■ A DISTINGUISHED FICTIONAL GENTLEMAN known as the Autocrat of the Breakfast Table once discoursed on the virtues of being conceited. "Conceit," said he, "is to the human character as salt is to the ocean; it keeps it sweet and renders it endurable." And he might have said the same regarding the character of various collective human activities, such as our various industries. I think none of us would care to be associated with an industry which had no right or desire to be conceited about itself.

All of you who are here today are associated with the concrete industry in some way; but like all men of broad interests you have followed progress in other fields as well. In some of those areas the rate of technical advancement has been no less than fantastic—at least, it appears so to those who are viewing such advancement from outside the inner circles. "What," you may then ask, "have we in the concrete construction field to offer in comparison to justify any expression of conceit over our own accomplishments? Even if we exclude the military fields," you might add (and I think we must exclude them, for problems related to our survival must be attacked from a unique viewpoint and with a unique research budget which exclude comparison), "consider communication and transportation; medicine; and organic chemistry and its myriads of plastics and resins with vivid colors, adaptability to any shape, and strengths that we dare not even dream of for concrete."

DYNAMICS

Many of us will confess that we view these seemingly esoteric activities in other fields with a degree of awe which may engender a sense of inferiority. You might well say to an aeronautical engineer, "You have accomplished something that to me, a concrete engineer, is beyond all comprehension; you have built a device which weighs 300,000 lb, and you have not only gotten it off the ground but you have flown it from Los Angeles to New York with a good payload in 4 hr." As a matter of fact I have had such discussions with aeronautical engineers, as doubtless many of you have, and the response is always about like this: "Oh, it's not so difficult. You use the laws of aerodynamics and the methods of stress analysis, and you make a lot of wind tunnel tests, and then you build the machine and give it a test flight. Then you bring it back to the shop and start correcting the mistakes you made. All this, of course, may require a million man-hours of engineering. If the machine is a really new and different type you will fly it for a few thousand hours before you trust it to take up passengers. But what gets me is how you fellows go about designing a building hundreds of feet high, different from any ever built before, that, on the first trial, must be absolutely safe against collapsing from its own weight or toppling over from wind loads or settling unequally on its foundations."

Industry experience

If the conversation continues we point out that the first book on reinforced concrete was published 80 years ago; so with that background it is quite in order that we should now be constructing concrete buildings, without resort to trial and error, 40 stories high (with 90 stories and more on the drafting boards), and building concrete shells spanning 720 ft, and vehicular bridges of prestressed concrete 24 miles long. Our modest aeronautical colleague will counter that the Wright brothers made a successful plane flight in 1905, and modern planes are therefore just a natural consequence of 50 years of development. Similarly, the electronics engineer sees nothing remarkable in the present hi-fi and TV sets, considering that the triode vacuum tube with its associated circuits was invented by De Forest in 1906; in fact, he is apt to be a bit disgruntled because the rate of progress has been so slow.

It has become a common belief that progress in structural engineering is made slowly. If we view the centuries that lie behind us, that certainly is true. If we view the decades immediately behind us it is, at the least, questionable.

Perhaps it has become apparent by now that a part of my theme at this time is that the concrete engineer or builder need not take a back seat in any discussion devoted to science and technology (again, perhaps, excluding those activities devoted to survival and to exploring outer space).

In our discussion with the hypothetical aeronauticist we mentioned a few concrete structures of outstanding dimensions; but we said nothing about the 20 or more types of cement which have been developed for special uses, nor about solving a complex problem of durability by entrainment of air bubbles, nor of special construction techniques which have made possible the most massive structures ever built by man. We said nothing about the myriad of problems which had to be solved each time that our construction material was put to new use; as in concrete building units, concrete pipe, concrete lining for tunnels and canals, concrete chimneys, piling, tanks, silos and storage elevators, light poles, insulating wall panels, thin shells, pavements and hangars for modern aircraft, precast or prestressed structural members, lightweight structural members, or radiation shields for nuclear energy. Nor did we mention the problems solved in connection with such construction techniques as slip form, lift slab, vacuum treatment, pneumatic application; nor the design problems dealing with resistance to earthquakes and atomic blasts.

ACI's contribution

We did not even mention another aspect of our industry of which we may well be proud—or even conceited. I am referring to the Institute which has honored me for the past year with its presidency, an organization which for more than half a century has played a vital role in a combined professional-industrial capacity in bringing about the progress of which I have spoken. Within its 55 volumes of *Proceedings* are recorded the great majority of the discoveries, the findings, and the scientific and constructional advances which are responsible for that progress.

Economical progress

Perhaps in comparing the concrete construction industry with some others we may indeed concede that we can set up no single accomplishment against a Boeing 707 nor even against a color TV set. But the over-all accomplishment in terms of improving our national welfare and of the creation of wealth through developing a relatively new construction material, and new construction techniques, is something for justifiable pride. But if we wish to be really conceited, rather than just complacently proud, perhaps we can resort to a subterfuge. Let us forget about progress in absolute terms, and think in terms of the cost of that progress as a fractional part of the dollar value of our product. We shall have the satisfaction of finding that, in those terms, the last few decades of progress in many other fields have cost 10 to 100 times what we have paid. Some might consider that a dubious claim to fame, and I shall say a little more about it later.

In case you are wondering why I am taking some pains to give our industry a pat on the back, I shall tell you; in the first place, the industry deserves it; and in the second place, the occasion of this address seems a good time to point out that the industry might also benefit from receiving the

front end of a shoe propelled at a good velocity and administered to that portion of the anatomy on which it spends entirely too much of its time reposing.

STATICS

It is a rather well known fact of life that in this ever changing world, progress is always about proportional to challenge. The rate of progress in our field has never been equaled in the past, but, even so, are we keeping up with the challenge? The answer must be something less than a positive yes.

A few years ago construction took away from agriculture the position of top industry in this country. In 1959 it will be a $50 billion industry. How many of these construction dollars are being reinvested in the industry to improve its products? The answer is, a small fraction of 1 percent; whereas other industries with technologies no less complex than ours find that to meet the challenge of today requires "plow-back" budgets—some call it research and development—of 3 to 5 percent, or even 10 percent, of gross sales.

Research needed

Just 20 years ago a strong and effective plea was published in the ACI JOURNAL to "shorten the lag between research and practice," on the basis that the time lag was too great between the discoveries and the findings of research in engineering and their acceptance and application by engineering practitioners. In the field of concrete construction, perhaps more than in any other field of engineering, we are on the verge of a reversal of that situation. Research is not occupying its time-honored place well ahead of practice, and in some areas it is definitely lagging behind. Examples might be cited by the dozen, but that can be done by each of you in his own field, and need not be recorded here. The fault cannot be ascribed to the research workers, and it is difficult even to say that the industry is at fault, for the industry is composed of many segments which encounter difficulty in getting together. We are confronted, rather, by an unsolved problem in methodology. ACI has been working on the problem, though not intensively, for years; the Reinforced Concrete Research Council has made some progress; the American Society of Civil Engineers has recently become acutely aware of it. The answer will come either from government (which is also working on it) or from industry. If from the latter, it will be through the leaders of our industry developing a greater awareness of the problem and then pooling their administrative, intellectual, and financial resources to solve it.

Technical partnership

Another area in which we are lax and in which, again, it is difficult to place the fault, is in the matter of collaboration between the architect, the structural engineer, and the contractor. In Europe the same organization frequently serves in all three capacities, but rarely so here. The need for inti-

mate collaboration is greater for concrete than for any other construction material. In the words of Mario Salvadori, referring to concrete structures, "Some of the most astounding results in modern architecture have been obtained by simply having the creativity of the engineer and the contractor, which is of a technological character, help the creativity of the architect, which is of an artistic character." In addition to the benefits of this sort of partnership in achieving the finest in architecture, it is certain that our billions of construction dollars would go further if the architect had at his disposal the contractor's intimate knowledge of cost, of construction equipment, erection methods, union regulations, and similar factors. I have repeatedly heard designers complain because a contractor bids a job successfully, and then proposes that the design be changed to fit existing equipment or methods. And the contractors complain because the design was made with no regard to their problems. But the owners are the ones who have the real complaint, because construction costs more than it should. The solution of that problem does not appear to be near-at-hand, but it must come. The USSR has made progress, but in ways which we would not find acceptable.

A look at the product

Closely related to the matter just discussed is a statement by architect Minoru Yamasaki which is well worth repeating: "My major indictment of the concrete industry is that it persists in thinking of its product as a *crude* material. No building should be built crudely; all good architecture throughout history has been fine and elegant. This can and should be attained with concrete, and with precast concrete both elegance and economy are possible." That statement needs no elaboration by me. It is an important pronouncement, and I trust it will be heeded.

On the subject of "what's wrong," one cannot possibly avoid mentioning the habitual use of outmoded specifications for concrete construction, carryovers from bygone days; but so much has been said on that subject that little remains. Sometimes the fault lies in a lack of authoritative information, which reverts to my first point, the need for more research; but more often, perhaps, it lies in a lack of initiative on the part of the specification writers which might be remedied by early collaboration with the contractors who, whatever their faults may be, cannot be accused of lack of initiative and imagination. And that reverts to my second point, lack of collaboration between architect, engineer, and contractor.

Uniform nonuniformity

Let me cite just one more item. We are now engaged in a $40 billion program of constructing an interstate and defense highway system. There is no reason to expect that a single set of design criteria for concrete highways should apply throughout the country, and such is certainly not the case. A recent survey by a government agency is interesting in that respect. The report on the survey notes that 5 of our states do not construct concrete pavements; 17 states construct only the nonreinforced type, 13 construct

only the reinforced type, and 14 construct both types. The distribution by types has little or no relationship to foundation or climatic conditions. Expansion joints in reinforced pavements, required in 10 states, are uniformly spaced at intervals ranging from 30 to 615 ft, with no two states using the same spacing. Expansion joints in nonreinforced pavements are required in 2 states, with spacings of 600 and 1200 ft. The spacing of contraction joints in reinforced pavements range from 15 to 106 ft, with only a few states using the same spacing. Contraction joint spacing in nonreinforced pavements ranges from 15 to 80 ft with most states using 15 or 20 ft. Reasons given by some states for their practices were found to be in direct conflict with reasons given by other states. The survey led to the conclusion, rather modestly stated, that "additional experimentation would seem desirable."

Perhaps I have been fortunate enough to mention some shortcoming within our industry about which the Institute will concern itself in the future. It is traditionally the task of the incoming president to take on his shoulders the burden of all problems left unsolved by the outgoing president. In recognition of that I wish to repeat at this time a small ceremony originated by one of my predecessors, and to hand to Prof. Phil M. Ferguson this token of his office, the eight ball.

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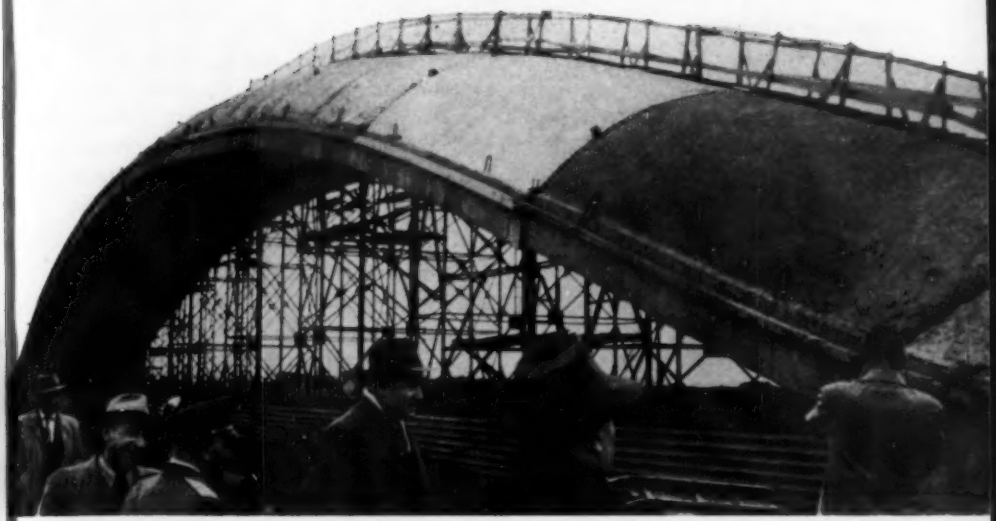


Russian Progress in Concrete Technology

JAMES D. PIPER and WALTER H. PRICE

In May 1958 the authors inspected concrete construction in Moscow and Leningrad where they visited laboratories, design offices, precasting plants, housing developments, and bridge and plant construction. They report that precast concrete is being used almost exclusively for building construction in these cities, where 86,000 apartments, mostly in five-story buildings, are planned for 1959. Prestressed concrete is used wherever possible in the Soviet Union and the Russians are engaged in large-scale research and development of concrete materials and construction.

IN THE SUMMER OF 1957, five Russian engineers were invited to attend the World Conference on Prestressed Concrete in San Francisco. These engineers in turn asked Prof. T. Y. Lin, one of the directors of the conference, to select a delegation of six American engineers to visit Russia as guests of the Soviet Academy of Construction and Architecture. The group was to have the opportunity to study Russian advances in concrete technology.

American delegation

In addition to the authors, the American delegation included: Prof. T. Y. Lin, University of California, Berkeley, chairman of the delegation; Prof. Boris Bresler, University of California, Berkeley; David P. Billington, Roberts and Schaefer Co., New York; and Ben C. Gerwick, Jr., of Ben C. Gerwick, Inc., San Francisco. The tour followed the Third International Congress on Prestressed Concrete, May 5-10, 1958, in Berlin, and was limited to the areas around Moscow and Leningrad.

A full schedule was arranged by the Academy. During 10 days in Russia, the delegation visited two research laboratories, five prestressing plants, five construction sites, and two design offices, and held meetings at the Academy of Construction and Architecture, at Moscow University, and at the office of the State Committee on Construction. We also presented papers on United States concrete practices to large groups in Moscow and Leningrad. Evenings were occupied by visits to the opera, ballet, Cinerama, circus, a football game, and by two banquets given in our honor.

Russian engineers

The engineers we met in Russia were capable and well informed. They were completely familiar with the latest developments in concrete technology in the United States. Many could speak English, and we were provided with an engineer-interpreter who was with us during every inspection trip. There were no restrictions on picture taking and we were furnished literature on many of the projects visited. Our hosts seemed anxious to show us every phase of concrete operations in the Moscow and Leningrad areas.

As vice-president for promotion of the Portland Cement Association, JAMES D. PIPER supervises all promotion activities of that organization, including the work of its regional and district offices serving 46 states, the District of Columbia, and British Columbia. An ACI member, Mr. Piper is a registered engineer and licensed architect. He served PCA in various engineering posts throughout the United States before assuming his present duties in Chicago in 1955.

●

WALTER H. PRICE, past president of the American Concrete Institute, heads the engineering laboratories of the U. S. Bureau of Reclamation at Denver. His work there includes materials testing and research problems related to the construction of large concrete dams and irrigation works. Long active in ACI committee work, and author of several JOURNAL papers, Mr. Price received the Institute's Wason Medal in 1951 for his paper "Factors Influencing Concrete Strength."

Fig. 1—Precast thin corrugated arch under test at Moscow laboratory

Other members of the delegation have published their views of the Russian trip (list at end of paper) and the delegation has prepared a combined report from which much of the information here presented has been taken.

Precast, prestressed construction increased

Since the end of World War II, there has been a rapid increase in the use of precast and/or prestressed concrete in Russia; in 1958 the construction program included 64,000 apartments in Moscow and 22,000 in Leningrad, plus many schools, hospitals, and office buildings, all of precast concrete. There are several reasons why precast concrete is favored in Russia. For one thing, large numbers of similar apartments can be built and their construction coordinated with production in the factory. Second, precasting methods are suited to the cold climate, permitting construction to progress through the winter months; and third, precast concrete saves steel. We were told that structural steel design units in Moscow have been abolished and concrete is to be used exclusively from now on for building construction.

RESEARCH

Although many of the apartment houses being constructed are similar, their design and construction are not completely standardized, and constant efforts are being made to improve materials, manufacturing processes, and construction procedures. We saw some of this development work in the large laboratories in Moscow, where 600 engineers and scientists are working three shifts a day on problems in the building field.

Full-size members tested

The Russians apparently prefer to test full-sized machines and structural members, and these laboratories are well equipped for testing large scale members. Tests were underway on full-size precasting and prestressing equipment and on large beams, slabs, arches, and trusses (Fig. 1). Strains in a truss 98 ft long were being measured while post-tensioning was being

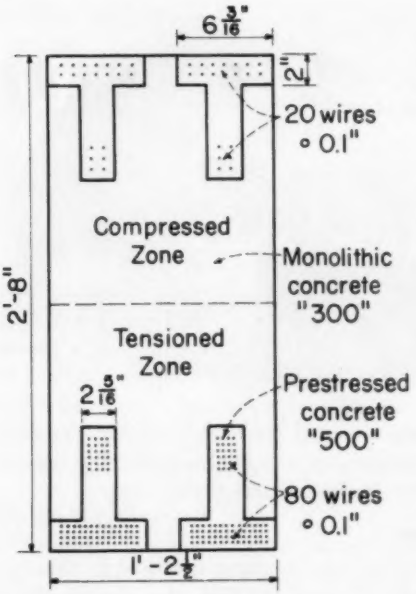


Fig. 2—Cross section of composite beam under test at Moscow laboratory. T-sections are highly prestressed

applied; this to be followed by tests under external loads. A composite member consisting of small T-shaped precast, prestressed beams cast in concrete to form a large beam was under test (Fig. 2). The small T-shaped beams were made of high strength concrete (about 7000 psi) and were highly prestressed. The concrete around the T-beams was weaker, about 4000 psi. It was claimed that excellent bond occurred between the two concretes and that this composite resulted in a saving of steel with better utilization of the tensile strength of the concrete. Crane rail beams with the rail connected to and figured as part of the beam; precast thin shell units of many types; precast, prestressed rail support to substitute for ties; and prestressed pipes were also being tested.

Expanding cement studied

The laboratory of the Research Institute of Concrete and Reinforced Concrete is doing some interesting work on an expanding cement for self-stressed concrete units. The cement expands after the concrete has hardened and developed bond placing the reinforcement in tension. The magnitude of expansion is affected by the composition of the cement, fineness of grinding, water-cement ratio, and method of curing. We saw 6 in. diameter pipe about 10 ft long being cast with expanding cement in the laboratory. As yet there has been no wide use of expanding cement in the USSR.

Other tests; materials studied in relation to durability

In addition, tests were underway on architectural concrete surfaces (Fig. 3), foam and lightweight concretes, silicones for surface protection, refractory concrete, and methods of improving the durability of concrete. The Russians apparently are completely familiar with retarders, air-entraining agents, accelerators, and pozzolans.

The Russians have found that where pozzolans are used to replace a portion of the cement, the resistance of the concrete to freezing and thawing is reduced. The use of pozzolans is therefore restricted to the interior of dams and to unexposed concrete.

Where heat is not available, concrete has been placed in Russia at temperatures as low as 0 F by adding 18½ percent calcium chloride and 4½

Fig. 3—Architectural surface for external precast slab seen in Moscow laboratory

percent sodium chloride, by weight of the water, to the mix. Such concrete has poor workability and low durability, and is used only where the concrete will be continuously submerged or continuously dry.

The advantages of entrained air in improving durability of the concrete is recognized, but there is some disagreement among the Russian engineers as to whether the length of winter protection can be reduced for air-entrained concrete as permitted under ACI 604-56.

Cement

About 33 million tons of cement were produced in Russia in 1958 under a single specification which includes portland-pozzolan and portland-slag cements. Specifications do not distinguish as to type and composition, and the cement is graded according to performance in physical tests. There are six grades of cement—200, 250, 300, 400, 500, and 600—which complicates cement storage and use. Grade 200 indicates that cubes made with that cement must have 200 kg per sq cm strength at 28 days, with similar meaning for the other grade numbers. The cubes are made with very dry mortar having a water-cement ratio of about 0.30 by weight.

PRECASTING PLANTS

The five precasting plants we visited—two in Moscow and three in Leningrad—were similar in over-all operation, but varied greatly in detailed methods and equipment. Generally, the aggregate arrives by railroad car or barge and is elevated from storage piles by enclosed conveyor belts to hoppers over the batching plant. The cement arrives by rail and is stored in silos at the plant. The plants are equipped with push button weighing controls. Tilting mixers of about 1.5 cu yd capacity are used. The concrete is conveyed by belt from the mixer to hoppers which travel on rails over the forms for casting the units. Methods of vibrating the concrete vary between plants, but generally compaction is obtained by vigorously vibrating the entire unit or table to which the form is attached.

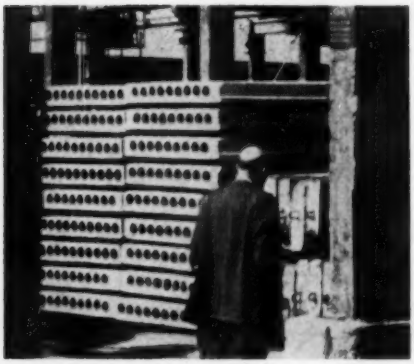


Fig. 4—Hollow core floor slabs in storage yard at Plant No. 6, Moscow

Natural sand and gravel graded up to about ¾-in. size was used in all plants. A uniform no-slump concrete was produced, containing about 670 lb of cement per cu yd, with a water-cement ratio of about 0.34 by weight. A strength of 2840 psi, as measured on 4-in. cubes, was required before the units could be shipped from the plant. Each plant had a small laboratory for control testing. One plant had facilities for testing full-size floor slabs and for developing casting equipment.

Prestressing practices

Hollow floor and wall slabs (Fig. 4) were the main products of the plants we visited. Prestressing of the floor slabs was accomplished by stretching a continuous high strength strand of wire around pegs set in the base of the slab form. The completed floor slabs were about 21 ft long, 9 in. thick and 4 or 6.5 ft wide with either circular or oval cores. Generally the circular holes were 6 in. in diameter at about 8-in. centers extended through the length of the slabs. Two main types of machines were used for this continuous prestressing: one, a turntable with a stationary feeder head (Fig. 5), and the other, a movable head with a stationary form. Each plant had facilities for fabricating welded wire mesh and for reinforcing units that were not prestressed.

Curing practices

The units were steam cured immediately after casting. One plant steam cured its products at 203 F, requiring 1 hr to raise the temperature and 1 hr to lower the temperature, with not less than 6 hr at 203 F. Other plants steam cured their products at 176 F with 3 hr for raising the temperature, 3 hr for cooling, and 10 hr at 173 F. There were many differences in the detailed methods and procedures used in the plants visited. We were told that the plant director was given a quota and certain standards to meet and that within these broad limits, he could operate the plant as he saw fit.

Equipment and quality of production

The plants were well equipped with tracks and cars for moving the products, hoists and traveling overhead cranes, vibrating tables, automatic prestressing machines, automatic welding machines for fabricating wire mesh, steam curing facilities, and storage yards. One of the batching and mixing plants we saw had a tile floor. It was completely automatic with dust collectors over the mixers and an electric moisture meter in the sand bins.

In general, the finished product was rough and the edges not true. In many slabs, cracks resulting from the casting operations extended to the steel. However, these products pass the strength requirements and after being covered by plaster or brick facing, they apparently perform as intended.

Plant and industry capacity

The plants operate on a three-shift basis and employ from 400 to 600 men. Production varies from about 100,000 cu yd to 150,000 cu yd per year. We were told that there were 25 precasting plants in Moscow with a combined output of 2,600,000 cu yd per year.

In 1957, the production of prestressed concrete in the USSR was given as 1,600,000 cu yd; total precast products with and without prestressing as 12,000,000 cu yd. By 1960 the annual production of prestressed concrete is set for 9,000,000 cu yd. and, for precast products 33,000,000 cu yd, indicating a rapid growth planned for this type of construction.

DESIGN

We visited two large design offices, one in Moscow and the other in Leningrad. The Moscow Design Institute designs and supervises all building construction in Moscow. It employs 3100 engineers, architects, and technicians, over half of whom are women. All design and construction are planned and scheduled 2 years ahead. Routine designs are simplified by standardization. Catalogs are prepared listing available precast and prestressed concrete units used in standard construction. We were given a catalog that covered nine types of precast concrete foundation units and a similar number of beams, columns, floor slabs, stairways, and roof units. These catalogs are constantly under revision to include improvements.

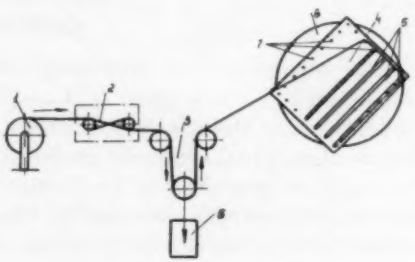


Fig. 5—Diagram of turntable type prestressing machine

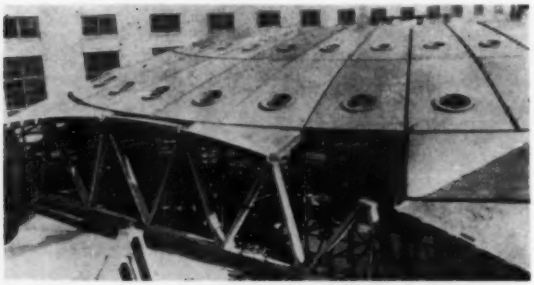


Fig. 6—Shell 131 x 131 ft containing 135 pieces was assembled on ground before being raised into position as roof of precasting plant

Occasionally the Moscow Design Institute designs special buildings. We were shown plans for a permanent building exposition in Moscow which will have an unobstructed roof span of 200 ft. The roof will be made of corrugated precast thin shells with a structural depth of 8 ft. The sections are to be precast in lengths of about 15 ft. and assembled on the job site.

Design Institute No. 1, in Leningrad, differs from the Moscow Design Institute in that it specializes in large civil engineering and industrial projects such as dams, hydroelectric plants, docks, hangars, and industrial buildings. Only about 10 percent of its work is for the city of Leningrad; the rest is for projects all over Russia as well as in China, Czechoslovakia, Poland, and Hungary. This institute employs 650 engineers and technicians and is at present doing design work for about $335 million of construction. Much of this work must be individually designed; however, an effort is made to standardize construction wherever possible. For example, work is underway toward standardization of precast units for construction of factory and industrial buildings and prestressed concrete transmission towers.

We were shown an unusual design for a stadium roof 500 x 820 ft, supported only on columns around its perimeter. This roof will be 6.3 in. thick and will be constructed of post-tensioned precast units.

We found that the Russians are using precast slabs in river bank protection and precast reinforced concrete forms in the construction of dams and other hydraulic works. Typical form panels are 66 ft long, 13 ft high, and 4 in. thick. The panel acts both as a form and as a facing after the structure has been completed. Large quantities of precast forms were used in the construction of Kuibyshev Hydroelectric Station on the Volga River.

CONSTRUCTION

The enlargement of precasting plants was the only industrial building construction we saw while in Russia. Apparently because of the crowded conditions in Moscow and Leningrad, industry is not being expanded in these cities. In Leningrad, at the Avtovo concrete factory which is being enlarged, we saw three large experimental buildings under construction. The roof of the first was composed of two shells, 131 ft square, made up of 135 precast elements and supported only at the corners. The roof of the second

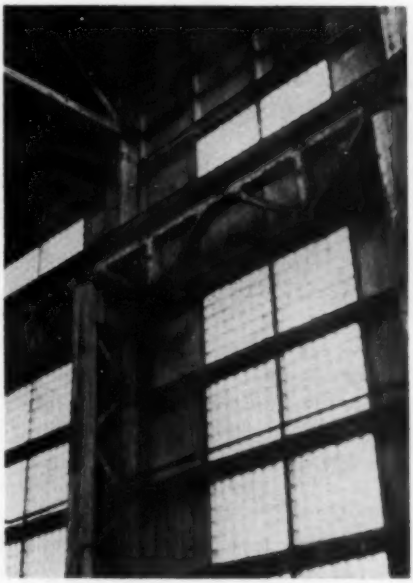


Fig. 7—Post-tensioned crane girder supported on precast lattice columns

was a huge 330-ft span thin shell arch composed of eight parallel units each 25 ft wide, separated by 8 ft wide skylights. The third was a large mill-type building made from built-up columns and trusses and composed entirely of precast elements.

The 135 pieces of 16 different shapes for the domes were assembled over scaffolding on the ground by welding, grouting, and post-tensioning, (Fig. 6). The four sides of the dome were supported by concrete trusses made up of a series of triangles with the top chord formed by the thickened edge of the shell. The assembled shell, weighing 560 tons, was lifted 66 ft by special jacks to a position slightly above its final height. Permanent columns, made up of precast hollow units each about 2 ft square by 3 ft high, were then erected under each corner of the shell and the shell lowered onto them.

The 330-ft arch shown on p. 1075 was made up of 13 precast elements bolted together on the ground as shown. The shell was 25 ft wide with a rise of 35 ft. It was approximately 6½ in. thick and weighed about 400 tons. The thrust of the arch was taken by cables encased in the corrugated precast elements shown in the picture (bottom). At the time of our visit no arch of this type had been lifted into its place in the roof.

The large mill-type building about 400 ft long by 100 ft wide rested on precast concrete spread footings. The columns were made of precast elements forming a lattice structure (Fig. 7). The roof was supported by slender precast, post-tensioned trusses 98 ft long, composed of five sections assembled on the job (Fig. 8). Post-tensioning wires ran through the bottom

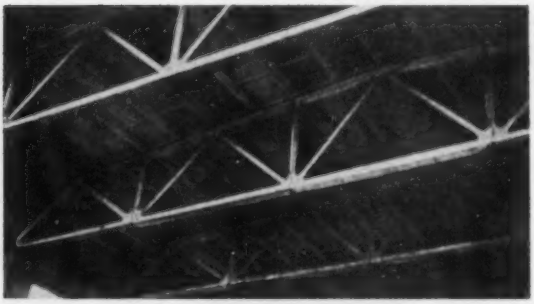


Fig. 8—Thin precast post-tensioned trusses used to support roof slab of mill-type building in Leningrad

chord. The trusses and columns were spaced at 20-ft centers. The crane girder was also a precast, post-tensioned truss supported on the columns about 30 ft above the floor (Fig. 7). The roof, walls, columns, footings, braces, and trusses of this building were all made of precast concrete. It is planned to use this building for the exclusive production of foam concrete.

Apartment houses

The apartment houses we saw under construction in Moscow and Leningrad were built almost entirely of precast and prestressed concrete elements. They were, in general, 5 stories high and were being built in blocks containing from 600 to 1000 apartment units. In addition to the apartment houses, a block included a central heating plant, a community store, a community meeting house, a kindergarten, and a nursery.

Each house contained from 60 to 80 apartments. A typical apartment had about 500 sq ft of space, and included two small bedrooms, a living room, bath, and kitchen. The plumbing fixtures were modern in appearance. The walls of the apartments were plastered and painted in pastel colors. The bedroom floors were covered with wood blocks about ½ in. thick and 6 in. square, while linoleum was used in the kitchen and bathroom, and in some cases the living room. The areas around the apartments were landscaped with shrubs, grass, and trees.

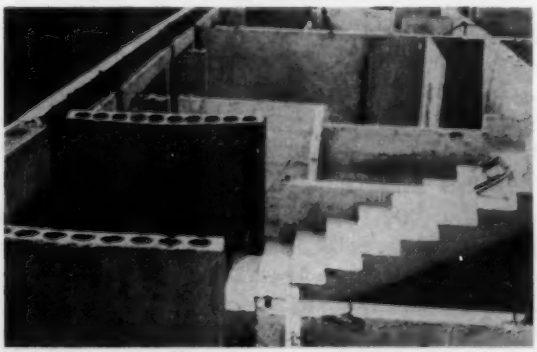


Fig. 9—Moscow apartment house construction. Walls and partitions serve as supporting members. Note hollow core wall slabs

Fig. 10—Completed apartments in Leningrad, faced with brick veneer cast on external slabs at factory

Two types of apartment houses have recently been constructed in Moscow and Leningrad: one, eight stories high using columns and beams as the supporting elements; the other, five stories high with walls and floor slabs as the supporting elements (Fig. 9). We saw only the latter type under construction. Hammerhead cranes with booms which could span the width of the building, and in some instances, gantry cranes which spanned the building, were used to move the precast pieces from delivery trucks to their position in the building.

The exterior walls, including the windows and doors of these apartments, were cast in panels about 12 in. thick and 10 ft square. The floors were made of hollow precast prestressed panels about 21 ft long, 4 or 6.5 ft wide, and 9 in. thick. Some of the nonbearing walls were made of slag cement concrete or gypsum. Metal pipes for heating were included in the partitions near the external walls. The horizontal mortar joints between the bearing walls were reinforced with a number of ¼-in. bars. The walls were fastened at the corners by welding protruding embedded metal together at three equally spaced points. Very little lateral bracing is provided, because Moscow and Leningrad are in nonseismic regions. Some of the apartment houses were faced with a 1-in. veneer of brick or tile cast on the slab at the factory (Fig. 10). The individual finished buildings had a neat appearance both inside and out, but the over-all impression in these huge housing developments was one of drabness.

Every effort was apparently being made to save steel. We noted glass tubing used as conduits for the electrical wiring, and in one development radiators of concrete were installed under the windows (Fig. 11).

Bridge in precast and prestressed concrete

We visited the site of a large bridge being built across the Moscow River to connect the university and the adjacent large new residential area with the main part of the city. The bridge will carry eight lanes of highway traffic on its upper level and two lanes of subway traffic on the lower level. A subway station will be located on the bridge. Including the approaches it will be 4000 ft long. The main span is built on a 37-deg skew and consists of an

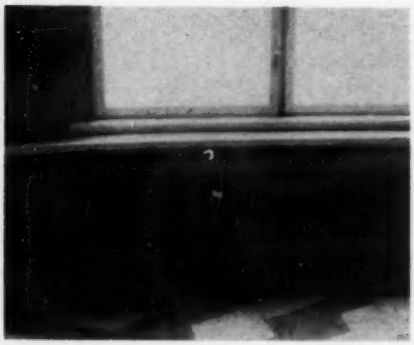


Fig. 11—Concrete radiator in new Moscow apartment

arch about 350 ft long flanked by two smaller arches approximately 150 ft long. The four parallel chords of the arch are made up of precast pieces connected by welding the projecting reinforcement and packing the space between with concrete. The thrust of the arch is taken by horizontal chords assembled from precast units and post-tensioned at the site. Half of the main span was being assembled on the bank of the river at the time of our visit. This assembly was moved sideways onto pontoons and floated into place over the supporting piers, where it was to be lowered into its final position (Fig. 12). Each assembly weighed over 5000 metric tons.

The approach spans were supported by precast post-tensioned trough-shaped beams about 72 ft long, 5 ft wide, and 4 ft deep, weighing 37 tons each. The girders supporting the columns were composed of two L-shaped sections tied to one another and to the top of the columns by concrete placed in a space provided for this purpose between sections. The columns were precast piles 16 in. square and from 30 to 70 ft long, driven to a design load capacity of 90 tons. The precast deck slabs were placed transversely and joined to the beams so that they acted compositely with them. The precast units used in the bridge were of much better quality than those used in housing construction. They appeared to be made of richer concrete, were true to line and dimension, and were free of patches and honeycomb.

Erection of all precast members in the bridge was being accomplished with huge gantry cranes running on tracks and spanning the entire construction. The crane spanning the work of the main arches probably had a span of 150 ft and a capacity of 50 tons. High towers with flood lights were in evidence indicating that this job was proceeding on a three-shift, 6-day week. This was the case in all the laboratories, factories, and construction we visited.

The construction of this bridge and the factories we saw being built in Leningrad indicate that the Russians are willing to experiment with new methods on a comparatively large scale. After a novel design has been worked out, it is then used in many places throughout the country.

CONCLUSION

The USSR is exerting great energy toward expansion of its cement and concrete industry. At present, Soviet cement production is about 60 percent that of the United States; the amount of concrete they produce is probably less than half that of the United States. This is due possibly to the poorer quality of some of their cement and to the richer mixes required by the colder climate. Their expansion of cement production is progressing at a faster rate than ours and it is claimed that they will meet our production capacity in not too many years.

The Soviet engineers were very much interested in our work in the United States and arranged for a series of lectures to be given by our delegation both in Moscow and Leningrad. These meetings were attended by an overflow crowd in each city, and many questions were asked following the lectures. On the last day of our visit, we met with V. A. Kutsherenko, deputy chairman of the Soviet Council of Ministers and chairman of the State Committee on Construction, which is in charge of all construction in the Soviet Union. He opened the meeting by saying that he did not want to hear our compliments but rather our criticisms. He informed us that he expected the quality of the precast products and the construction on the job to improve now that they had developed trained men in this rapidly expanding industry. He informed us of their plans to greatly increase the use of precast and prestressed concrete during the next 7 years. He said that an entire institute which had been designing steel structures had recently been ordered to design only concrete structures from now on.

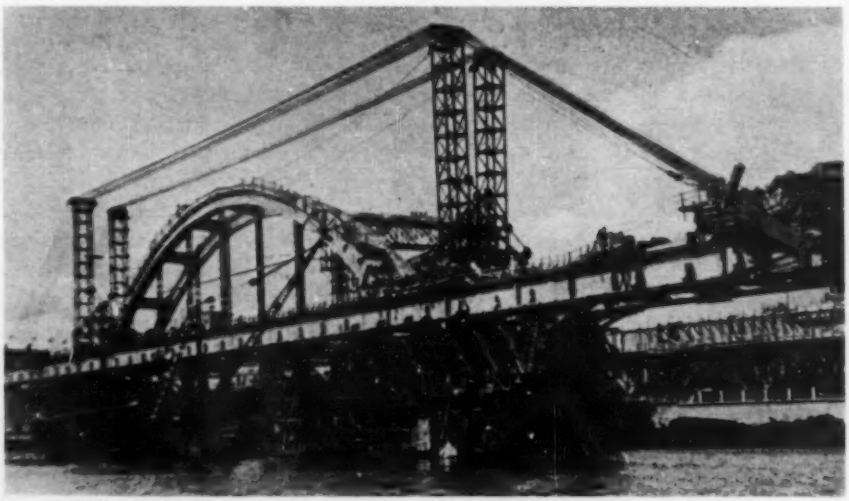


Fig. 12—Floating a bridge on the Moscow River into position

Our time while in Russia was completely scheduled from morning to midnight each day. In addition to the engineering projects, we were conducted on tours through the Kremlin, the 40,000-room University of Moscow, the famous museums of Leningrad, the grounds of the winter palace of the czars in Leningrad, and on an excursion into the country near Leningrad. The evenings were occupied with opera, ballet, circus, and banquets. Our trip ended where it began, at the Moscow airport, after 10 interesting days of activity.

ADDITIONAL INFORMATION

Published articles by members of the American delegation describing the Russian tour include:

1. Billington, David P., "An American Engineer Views Precast and Prestressed Concrete in the Soviet Union," *Civil Engineering*, V. 28, No. 10, Oct. 1958, pp. 734-739.
2. Gerwick, Ben C., Jr., and Lin, T. Y., "Prestressed Concrete in Russia," *Journal*, Prestressed Concrete Institute, Dec. 1958.
3. Lin, T. Y., "Russian Prestressing Methods Unique," *Concrete Products*. V. 61, No. 10, Oct. 1958, pp. 28-34.
4. Piper, James D., "U. S. Engineer Views Soviet Construction," *American Engineer*, Sept. 1958.

Presented at the ACI 11th regional meeting, Detroit, Mich., Oct. 27, 1958. Title No. 55-65 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 10, Apr. 1959 (Proceedings V. 55). Separate prints are available at 50 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by July 1, 1959, for publication in the Part 2, December 1959 JOURNAL.

Title No. 55-66

Shear Strength of Two-Span Continuous Reinforced Concrete Beams

By

JOSÉ J. RODRIGUEZ, ALBERT C. BIANCHINI,
IVAN M. VIEST, and CLYDE E. KESLER

FIFTY-TWO CONTINUOUS reinforced concrete beams were tested under concentrated loads and the results analyzed in an effort to determine the following: (1) effect of continuity on the shear strength of statically indeterminate members, (2) contribution of web reinforcement to shear strength, and (3) to establish the minimum amount of web reinforcement required to prevent shear failures. The following variables were included: type of loading, length of negative moment longitudinal reinforcement, percentage of web reinforcement, and grade of longitudinal reinforcement.

Fifteen beams were designed with no web reinforcement, 13 beams had the amount of web reinforcement required by the 1951 ACI Code, and 24 beams were designed with more web reinforcement than required by the ACI Code.

An analysis of the test results indicated good agreement with the calculated values for the loads at diagonal tension cracking, shear compression failure, and flexural failure, as computed with the aid of existing mathematical expressions developed from tests of simple and restrained beams. When the amount of web reinforcement was determined by equating the formulas for maximum flexural and maximum shear loads, it was found to be more than was necessary.

CURRENT DESIGN METHODS for proportioning reinforced concrete members to resist shear stresses take into account the external shear forces, cross-sectional dimensions, concrete strength, and the amount of web reinforcement. However, over 50 years ago Talbot[1] demonstrated that the shear behavior of concrete members is affected also by the ratio of span length to effective depth of the member, and by the percentage of longitudinal reinforcement. More recent investigations[2-8] have shown that these factors neglected in shear design have a major effect on shear behavior of reinforced concrete members.

Most shear tests have been made on simple beams or on simply supported beams with overhangs. Application of the results of such tests to the design of continuous members involves the assumption that continuity has no effect on shear strength. Because the majority of beams are continuous in actual structures, it was considered necessary to determine to what extent this assumption is correct, or whether there are factors in continuity that have an effect on the shear behavior of statically indeterminate members.

JOSÉ J. RODRIGUEZ, instructor in the Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, has been active in his department's intensive study of the shear behavior of reinforced concrete beams. He received his MS at Illinois in 1958, and is a member of ACI, ASCE, and Sigma Xi. • ALBERT C. BIANCHINI, assistant professor in the same department, has participated in a number of concrete research studies dealing with welded wire fabric, shear strength, cracking, and the effect of slab concrete strength on column strength. Professor Bianchini, who received his MS in 1952, is a member of the American Concrete Institute and Sigma Xi. • ACI member IVAN M. VIEST is bridge research engineer, AASHO Road Test, Ottawa, Ill. While at the University of Illinois he conceived the project on which this paper is based, obtained sponsorship, and directed initial phases of the study. Dr. Viest is currently chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and a member of ACI-ASCE Committee 326, Shear and Diagonal Tension. A recipient of the Institute's Wason medal in 1955, he also received the ASCE Research Award in 1958. • CLYDE E. KESLER, professor of theoretical and applied mechanics, University of Illinois, has published numerous papers describing research in shear, fatigue, creep, and sonic behavior of concrete. In 1958 he was corecipient of the ASTM Sanford E. Thompson Award for a paper on creep. Professor Kesler's most recent ACI JOURNAL contributions were two papers in the fatigue symposium published last August. An ACI member since 1947, he serves on the Technical Activities Committee and several technical committees including: 209, Volume Change and Plastic Flow; and 215, Fatigue of Concrete.

Object and scope

The primary object of this investigation was to determine the effect of continuity on the shear strength of statically indeterminate beams. Two-span continuous beams were selected for their relative simplicity. The following variables are included: cutoff or extended longitudinal steel (see definitions), percentage and spacing of web reinforcement, type of loading, and grade of longitudinal reinforcement.

The first part of the investigation, which includes a total of 28 beams in Series N and Series A, was conducted in 1954-1955. It was concerned with the shear strength of reinforced concrete beams with and without web reinforcement. Beams with web reinforcement were designed in accordance with the 1951 "Building Code Requirements for Reinforced Concrete (ACI 318-51)."[9] The second part, which includes 24 beams in Series H and I was conducted in 1956-1957; it was concerned with establishing the minimum amount of web reinforcement required to prevent shear failure.

All specimens were provided in the end shear spans (Shear Span 1) with an amount of web reinforcement much larger than that required by ACI 318-51, to eliminate any possibility of shear failure in the span (Fig. 1).

After completion of the above investigation four additional beams, with an increase in cover thickness, were tested to observe whether increased cover thickness would prevent a splitting type of failure which occurred in a number of beams in the main investigation. Results obtained are summarized in the Appendix (p. 1128).

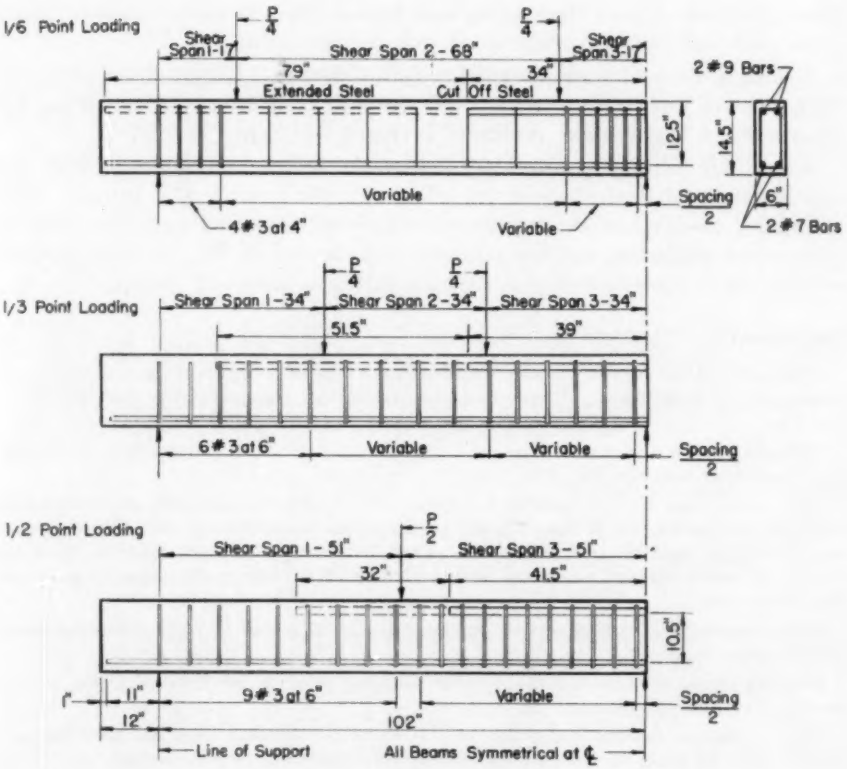


Fig. 1—Details of beams

Outline of tests

The investigation included tests of 52 two-span continuous reinforced concrete beams. Three different types of loading were used as shown in Table 1 and in Fig. 1. Table 1 also includes the amount of web reinforcement each beam had in Shear Spans 2 and 3, the type of negative moment longitudinal reinforcement, and the grade of longitudinal reinforcement.

Series N beams had no web reinforcement in either Shear Spans 2 or 3. Beams in Series A had the amount of web reinforcement in Shear Spans 2 and 3 in accordance with the requirements of ACI 318-51. In both series the longitudinal steel was of intermediate grade.

Except for two beams, all failures in Series N and Series A resulted from excessive shear. An extrapolation of the test data for these series was used to estimate the minimum percentage of web reinforcement needed to insure flexural failures in beams with intermediate grade longitudinal steel. This web reinforcement was used in beams of Series H; a higher yield point (hard

grade) was selected for the longitudinal steel of Series H to insure shear failure even with the higher percentage of web reinforcement.

Beams in Series I were designed to fail in flexure. Longitudinal reinforcement was of intermediate grade, and the web reinforcement was chosen by interpolation between the results of Series A and Series H.

In all four series, approximately half of the beams were designed with the top longitudinal reinforcement cut off in accordance with ACI 318-51. The remaining beams had the top longitudinal steel extended past the point of theoretical maximum positive moment as indicated in Fig. 1. The purpose was to study the effect of this variable in the behavior of beams.

Definitions

Cutoff steel is the negative moment reinforcement cut off according to the minimum requirements of ACI 318-51. These requirements were not changed in the 1956 edition of the Code.

Extended steel is the negative moment reinforcement extended beyond the permissible cutoff points as shown in Fig. 1.

Shear span is the distance between a support and the adjacent load point or between two adjacent load points. Shear Span 1 is the portion of the beam between the exterior support and the nearest load, Shear Span 2 is the portion between two adjacent loads on the same side of the center support, and Shear Span 3 is the portion between the center support and the nearest load.

Critical section is a vertical section through the edge of a load or support bearing block inside a shear span.

Diagonal tension crack is an inclined crack extending from the tension steel to the critical section.

Diagonal tension cracking load is that load at which the diagonal crack has extended sufficiently into the beam to cause an appreciable redistribution of internal stresses.

Diagonal tension failure occurs simultaneously with the formation of a major diagonal tension crack, regardless of whether other diagonal tension cracks had previously formed in the beam or not.

Shear compression failure occurs by crushing of the concrete in compression at the compression end of an existing diagonal tension crack.

Splitting bond failure is a combination of splitting along the steel caused by dowel action of the reinforcement and of bond failure caused by high longitudinal stresses in the steel.

Flexural failure occurs by crushing of concrete in the compression zone in the region of maximum positive or negative moment after both the bottom reinforcement in the span and the top reinforcement over the support have yielded.

Notations

Distances

a = M/V; effective length of the shear span equal to distance from the section of zero moment to a critical section (Fig. 2)

a' = distance between the critical sections at the opposite ends of the same shear span (Fig. 2)

a_c = $(M/V)_c$; distance from the section of zero moment to the intersection of a diagonal tension crack with the tensile reinforcement (Fig. 2)

b = width of beam

d = effective depth of tension reinforcement (Fig. 2)

TABLE 1—OUTLINE OF BEAMS

Type of loading	Series	Web reinforcement				Negative moment reinforcement	Grade of longitudinal reinforcement	Beam designation†
		Shear Span 2		Shear Span 3				
		Size and spacing	ρ^* percent	Size and spacing	ρ^* percent			
1/6-point	N	—	0	—	0	Extended	Intermediate	E6N1
		—	0	—	0			E6N2
		—	0	—	0			E6N3
		—	0	—	0	Cut off		C6N1
		—	0	—	0			C6N2
		—	0	—	0			C6N3
	A	—	0	# 3 @ 2¾ in.	1.33	Extended	Intermediate	E6A1
		—	0	# 3 @ 2¾ in.	1.33			E6A2
		—	0	# 3 @ 2¾ in.	1.33	Cut off		C6A1
		—	0	# 3 @ 2¾ in.	1.33			C6A2
	H	No. 10 wire @ 12 in.	0.039	# 4 @ 3 in.	2.22	Extended	Hard grade	E6H1
		No. 10 wire @ 12 in.	0.039	# 4 @ 4 in.	1.67			E6H2
		No. 10 wire @ 12 in.	0.039	# 4 @ 3 in.	2.22	Cut off		C6H1
		No. 10 wire @ 12 in.	0.039	# 4 @ 4 in.	1.67			C6H2
	I	No. 10 wire @ 12 in.	0.039	# 4 @ 3¾ in.	1.78	Extended	Intermediate	E6I1
		No. 10 wire @ 12 in.	0.039	# 4 @ 3 in.	2.22			E6I2
		No. 10 wire @ 12 in.	0.039	# 4 @ 3¾ in.	1.78	Cut off		C6I1
		No. 10 wire @ 12 in.	0.039	# 4 @ 3 in.	2.22			C6I2
1/3-point	N	—	0	—	0	Extended	Intermediate	E3N1
		—	0	—	0			E3N2
		—	0	—	0	Cut off		C3N1
		—	0	—	0			C3N2
	A	—	0	# 3 @ 6 in.	0.61	Extended	Intermediate	E3A1
		—	0	# 3 @ 6 in.	0.61			E3A2
		—	0	# 3 @ 6 in.	0.61	Cut off		C3A1
		—	0	# 3 @ 6 in.	0.61			C3A2
	H	# 2 @ 10 in.	0.17	# 4 @ 6 in.	1.11	Extended	Hard grade	E3H1
		# 2 @ 10 in.	0.17	# 4 @ 7½ in.	0.89			E3H2
		# 2 @ 10 in.	0.17	# 4 @ 6 in.	1.11	Cut off		C3H1
		# 2 @ 10 in.	0.17	# 4 @ 7½ in.	0.89			C3H2
	I	# 2 @ 10 in.	0.17	# 4 @ 7 in.	0.95	Extended	Intermediate	E3I1
		# 2 @ 10 in.	0.17	# 4 @ 6 in.	1.11			E3I2
		# 2 @ 10 in.	0.17	# 4 @ 7 in.	0.95	Cut off		C3I1
		# 2 @ 10 in.	0.17	# 4 @ 6 in.	1.11			C3I2
1/2-point	N			—	0	Extended	Intermediate	E2N1
				—	0			E2N2
				—	0			E2N3
				—	0	Cut off		C2N1
				—	0			C2N2
	A			# 3 @ 10 in.	0.37	Extended	Intermediate	E2A1
				# 3 @ 10 in.	0.37			E2A2
				# 3 @ 10 in.	0.37			E2A3
				# 3 @ 10 in.	0.37	Cut off		C2A1
				# 3 @ 10 in.	0.37			C2A2
	H			# 3 @ 4½ in.	0.82	Extended	Hard grade	E2H1
				# 3 @ 7 in.	0.52			E2H2
				# 3 @ 4½ in.	0.82	Cut off		C2H1
				# 3 @ 7 in.	0.52			C2H2
	I			# 3 @ 4½ in.	0.82	Extended	Intermediate	E2I1
				# 3 @ 5½ in.	0.67			E2I2
				# 3 @ 4½ in.	0.82	Cut off		C2I1
				# 3 @ 5½ in.	0.67			C2I2

* $\rho = \dfrac{\text{area of stirrup}}{\text{width of beam} \times \text{spacing of stirrups}}$

†The designation of each beam consists of two capital letters and two numerals. The first letter designates the type of negative reinforcement (E = extended, C = cut off); the first numeral designates the type of loading (6 = 1/6-point; 3 = 1/3-point; 2 = 1/2-point); the second letter designates the series indicating the amount of web reinforcement adjacent to center support in Series N and Series A (N = none, A = in accordance with ACI Code) and the grade of longitudinal reinforcement in Series H and Series I (H = hard grade; I = intermediate grade). The second numeral is added to distinguish between companion specimens (1,2,3) that in Series H and Series I vary only in the amount of web reinforcement in Shear Span 3.

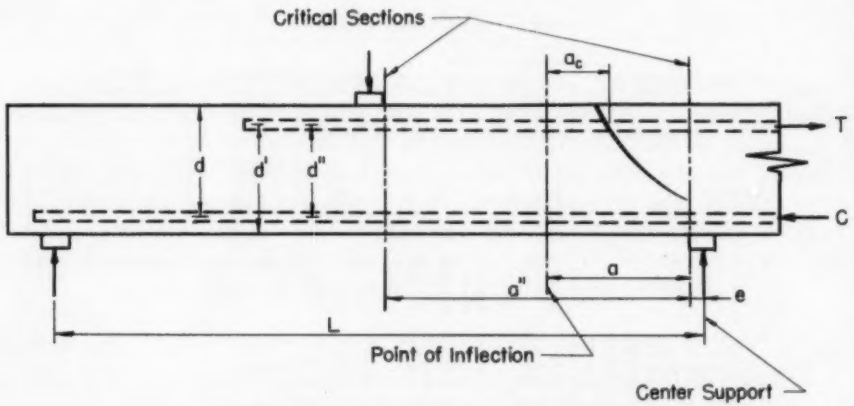


Fig. 2—Explanation of some of the notations

d' = effective depth of compression reinforcement (Fig. 2)

d'' = distance between the centroids of the tension and compression reinforcement (Fig. 2)

d_{min} = effective depth of tension reinforcement at the section of minimum moment

d_{max} = effective depth of tension reinforcement at the section of maximum moment

e = distance from theoretical point of application of a load or reaction to the edge of the loading block (Fig. 2)

L = span length (Fig. 2)

s = longitudinal spacing of web reinforcement

Stresses

f_c' = compressive strength of a 6 x 12-in. concrete cylinder

f_s = tension steel stress at failure

f_s' = compression steel stress at failure

f_y = yield point stress of tension reinforcement

f_y' = yield point of stress of compression reinforcement

f_{vy} = yield point stress of web reinforcement

v_c = nominal shearing stress at the intersection of a diagonal tension crack with the tensile reinforcement

v_u = nominal shearing stress at failure

Strains

ϵ_u = maximum concrete strain at failure

Moments and forces

M/V = moment to shear ratio at the critical section

$(M/V)_c$ = moment to shear ratio at the critical section of a diagonal tension crack

$M+$ = flexural moment capacity of a positive moment section

$M-$ = flexural moment capacity of a negative moment section

M_f = flexural capacity of a beam cross section

M_{min} = the numerically smaller of the two moments in a shear span with opposite-sign moments at the two critical sections; it should always be taken as a positive quantity

M_{max} = the numerically larger of the two moments in a shear span with opposite-sign moments at the two critical sections; it should always be taken as a positive quantity

M_s = shear moment capacity of a beam with web reinforcement

M_s' = shear moment capacity of a beam without web reinforcement

P = total load applied to the beam

P_f = total ultimate flexural load

V_c = external shear at diagonal tension cracking load

V_u = external shear at failure load

Constants, parameters, and ratios

A_s = area of tension reinforcement

A_s' = area of compression reinforcement

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

k_1, k_2, and k_3 = coefficients defining magnitude and positions of the internal compressive force in concrete compressive zone

$$n = \frac{E_s}{E_c}$$

$$N = 1 + \frac{M_{min}}{M_{max}} \times \frac{d_{max}}{d_{min}}$$

$$p = \frac{A_s}{bd}$$

$$\bar{q} = \frac{A_s f_y - A_s' f_s'}{bdf_c'}$$

$$r = \frac{\text{area of one stirrup}}{bs}$$

$$Z = 1 - \frac{N-1}{N} \times \frac{d''}{d}$$

SPECIMENS AND MATERIALS

Description of specimens

The details and dimensions of the beams tested are shown in Fig. 1 and Tables 1 and 2. All beams had a nearly constant rectangular cross section of 6 x 14.5 in. and an over-all length of 19 ft. Both spans were 8.5 ft long.

The longitudinal bottom reinforcement consisted of two #7 deformed bars which furnished approximately 1.6 percent of steel. They were continuous through both spans. An ACI balanced design would require 1.58 percent of longitudinal bottom reinforcement.

According to the elastic theory of structures, the ratio between the negative and positive moments is 1 58, 1.50, and 1.20 for the ⅙-point, ⅓-point, and ½-point loadings, respectively. Since it was considered undesirable to vary the amount of longitudinal reinforcement with the type of loading, two #9 bars, furnishing an area equal to 1.67 times that of the bottom reinforcement, were chosen for the top longitudinal steel. The top reinforcement was located over the center support. In beams with cutoff longitudinal steel, it was terminated 12 bar diameters beyond the theoretical point of contraflexure. In beams with extended reinforcement, the top reinforcement was cut off 20 bar diameters beyond the point of maximum positive moment.

Concrete cover and spacing of the longitudinal reinforcement were the same in all beams; these are given in Table 3. The values permitted by ACI 318-51 are also listed in Table 3.

All stirrups were of the closed type as shown in Fig. 1. Stirrups were made of #2, #3, and #4 bars and of No. 10 wire, depending on the shear span and the type of loading (Table 1).

Web reinforcement

In the end shear spans (Shear Spans 1) all specimens were provided with an amount of web reinforcement much larger than that required by the ACI Code (318-51), to eliminate any possibility of shear failures in the span (Fig. 1). According to the Code no web reinforcement was required in Shear Spans 2.

The specimens in Series N (e.g., E6N1) had no web reinforcement in Shear Spans 3. Beams in Series A (e.g., E6A1) were provided in Shear Spans 3 with the amounts of web reinforcement required by ACI 318-51. The Code required percentages of web reinforcement equal to 1.32, 0.60, and 0.34 for ⅙-point, ⅓-point, and ½-point loadings, respectively. ACI 318-56[9a] requires that the percentage of web reinforcement in Shear Spans 3 be 1.23, 0.75, and 0.58 for ⅙-point, ⅓-point, and ½-point loading, respectively. It also requires that web reinforcement be continued into Shear Spans 2, 18 in. and 6 in. for ⅙-point and ⅓-point loading, respectively. All other design limits required by ACI 318-56 give the same value as those of ACI 318-51.

TABLE 2—PROPERTIES OF BEAMS*

Beams	Width, in.	Effective depth of reinforcement, in.		Concrete cylinder strength† f'_c, psi		Yield point of reinforcement, psi		
						Longitudinal		Web, Shear Span 3
		Bottom	Top	Batch 1 and 2	Batch 3	Bottom	Top	
Series N								
E6N1	6.00	12.63	12.50	3090	3460	45,400	42,900	
E6N2	6.06	12.75	12.50	2640	2600	46,800	44,500	
E6N3	6.06	12.75	12.50	3290	3250	46,800	44,500	
C6N1	6.13	12.75	12.63	3790	3340	46,400	46,500	
C6N2	6.00	12.88	12.50	3210	2470	46,700	45,500	
C6N3	6.06	12.75	12.38	3150	3160	46,700	45,500	
E3N1	6.00	12.75	12.50	3650	3910	45,500	43,500	
E3N2	6.00	12.75	12.50	3380	3440	46,100	44,000	
C3N1	6.00	12.75	12.50	3350	3260	46,700	45,900	
C3N2	6.06	12.63	12.50	3290	3310	46,600	45,700	
E2N1	6.19	13.00	12.50	4200	4390	45,600	43,500	
E2N2	6.00	12.75	12.50	3010	2350	47,100	43,500	
E2N3	6.13	12.56	12.63	3330	3120	47,100	43,500	
C2N1	6.13	12.75	12.50	3780	3540	48,000	43,500	
C2N2	6.00	12.63	12.50	2700	2960	46,700	46,100	
Series A								
E6A1	6.00	12.63	12.50	3380	3570	45,400	44,500	50,700
E6A2	6.13	12.50	15.50	3200	2980	48,100	44,300	50,800
C6A1	6.00	12.56	12.50	3480	3090	46,100	45,300	50,000
C6A2	6.06	12.56	12.44	3170	3390	47,500	46,200	51,200
E3A1	6.00	12.50	12.44	3280	3650	46,100	43,600	50,600
E3A2	6.13	12.50	12.50	3160	3220	47,600	43,300	50,800
C3A1	6.13	12.50	12.50	3510	3260	46,100	43,500	50,400
C3A2	6.13	12.75	12.38	3410	3140	47,600	45,200	50,500
E2A1	6.00	12.50	12.50	3760	3590	46,400	45,600	50,200
E2A2	6.00	12.63	12.50	2830	2720	47,200	44,500	50,300
E2A3	6.13	12.63	12.44	2940	2860	48,000	47,400	50,900
C2A1	6.06	12.50	12.50	3260	3310	46,100	44,300	51,500
C2A2	6.19	12.63	12.25	3260	3090	47,000	45,000	50,600
Series H								
E6H1	6.00	12.80	12.30	3430	3500	54,500	58,600	46,200
E6H2	6.00	12.56	12.44	4190	3670	55,600	61,500	46,000
C6H1	6.00	12.87	12.83	3460	3480	56,100	58,200	47,300
C6H2	6.00	12.50	12.37	3860	3410	57,200	60,000	45,000
E3H1	6.00	12.53	12.00	3410	3970	55,100	58,800	48,000
E3H2	6.00	12.26	12.83	4170	3630	54,700	57,600	45,800
C3H1	6.00	12.52	12.45	3450	2930	55,800	60,000	46,100
C3H2	6.00	12.70	12.40	3500	2920	59,400	59,800	46,100
E2H1	6.00	12.57	12.77	3370	3020	55,100	58,300	50,600
E2H2	6.00	12.95	12.15	3310	2950	56,900	60,000	52,600
C2H1	6.00	12.51	12.25	3240	3150	54,700	58,800	51,300
C2H2	6.00	13.35	12.80	4320	3630	56,400	58,100	51,900
Series I								
E6I1	6.00	12.77	12.30	3860	3680	48,100	44,300	46,200
E6I2	6.00	12.74	12.30	3410	3310	49,000	44,000	45,900
C6I1	6.00	12.60	12.40	4040	3510	49,100	43,300	46,500
C6I2	6.00	12.47	12.52	3100	2970	49,700	42,900	46,700
E3I1	6.00	13.00	12.20	4130	3830	48,200	42,300	46,300
E3I2	6.00	12.40	12.25	3750	3240	48,900	44,600	47,100
C3I1	6.00	12.90	12.25	3720	3880	49,200	44,000	46,100
C3I2	6.00	12.42	12.40	2990	3120	47,800	45,700	46,600
E2I1	6.00	12.75	12.45	3770	3350	47,400	44,000	51,100
E2I2	6.00	12.75	12.35	3560	3390	47,200	45,700	51,000
C2I1	6.00	12.57	12.27	4150	3800	49,400	42,300	51,000
C2I2	6.00	12.70	12.12	2790	2630	48,600	44,600	51,700

*All beams were 14.5 in. deep and continuous over two 8.5-ft spans. The bottom reinforcement consisted of two #7 bars, the top reinforcement of two #9 bars, and the web reinforcement consisted of vertical stirrups of different sizes. Beams subjected to the same type of loading had the same web reinforcement between the end supports and the first load (Shear Span 1), and between two loads in the same span (Shear Span 2) for beams in Series H and Series I; no web reinforcement was provided in Shear Span 2 for beams of Series N and Series A; the web reinforcement varied for different beams between the inner loads and the center support (Shear Span 3) for beams of all series except Series N, which had no web reinforcement in Shear Span 3. The positioning of the reinforcement is shown in Fig. 1.

†Average of four to five cylinders from each batch.

TABLE 3—COVER AND SPACING OF LONGITUDINAL STEEL*

Item	Positive reinforcement		Negative reinforcement	
	Nominal	Minimum ACI 318-51	Nominal	Minimum ACI 318-51
Vertical cover, in.	1.5	1.5	1.5	1.5
Side cover, in.	1.5	1.5	1.25	1.5
Clear spacing, in.	1.25	1.33	1.25	1.33

*The cover dimensions are given to the outside face of the longitudinal reinforcement.

Results of Series N and Series A indicated that to prevent shear failures in Shear Spans 2 web reinforcement was needed in those spans. Therefore all beams in Series H and Series I were provided with enough web reinforcement in Shear Spans 2 to prevent failure.

The analysis used in this investigation for the shear compression strength of reinforced concrete beams is based on the hypothesis of an existing limiting shear moment capacity. Since the maximum moment in the continuous beams occurred next to the center support, the amount of web reinforcement in Shear Spans 3 was varied in corresponding specimens in Series H and I in order to determine the contribution of web reinforcement to the shear strength of beams.

Materials and mixing

Cement—A standard brand of Type I portland cement used throughout the tests was purchased in paper bags and stored under proper conditions. For Series N and Series A of the investigation the cement was purchased in three lots, and for Series H and Series I in two lots.

Aggregate—The fine aggregate used was a Wabash River torpedo sand. Sand used in Series N and A had an average fineness modulus of 2.94, while that used in Series H and I had an average fineness modulus of 3.24. The coarse aggregate was a Wabash River gravel of 1-in. maximum size. The average grading of the aggregates is given in Table 4. Absorption of both fine and coarse aggregate was about 1 percent by weight of surface dry aggregate.

Proportions and mixing—All concrete was proportioned for a nominal compressive strength of 3500 psi at 28 days. The proportions of cement to sand to gravel by weight were 1:3.50: 4.90 for Series N and A, and 1:3.84:5.76 for Series H and I. The cement-water ratios used in Series N and A, and in Series H and I were 1.3 and 1.11, respectively. Slump varied from 2 to 4 in.

TABLE 4—GRADING OF SAND AND GRAVEL

Sieve size	Series N and Series A, percent retained*		Series H and Series I, percent retained*	
	Sand	Gravel	Sand	Gravel
1½ in.	0	0	0	0
¾ in.	0	23.2	0	32.1
⅜ in.	0	73.8	0	76.9
No. 4	1.2	97.8	1.9	99.0
8	11.7	100.0	18.1	99.5
16	28.4	100.0	41.0	99.5
30	61.7	100.0	69.3	100.0
50	92.5	100.0	93.8	100.0
100	98.4	100.0	98.5	100.0
Fineness modulus	2.94	6.95	3.25	7.07

*Average for four lots of sand and gravel.

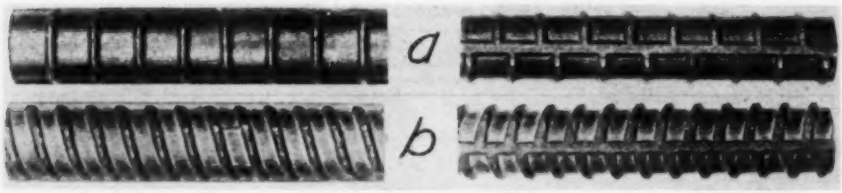


Fig. 3—Bar deformations

Materials were mixed in a nontilting, horizontal 6.5 cu ft drum mixer for approximately 5 min. Table 2 contains the average compressive strengths of concrete determined for each beam from tests of control cylinders. An average value is reported for Batches 1 and 2 corresponding to the concrete placed in the outside thirds of the beam, and a separate value for Batch 3 corresponding to the concrete in the middle third of the beam. In addition, initial modulus of elasticity was determined for Batch 3. The results were in good agreement with the values given by Lyse's equation:

$$E_c = 460\, f_c' + 1{,}800{,}000$$

Reinforcement—The longitudinal reinforcement consisted of #7 and #9 deformed bars of intermediate and of hard grade steel. The web reinforcement was made of #2, #3, and #4 bars and of No. 10 wire. The #2 bars and No. 10 wire had plain surfaces. All other bars sizes had deformations perpendicular to the bar axis (Fig. 3a), except for the #3 bars indicated in Table 5, which had inclined deformations (Fig. 3b). Bar dimensions (Table 5) were determined from five bars of each size. All bar sizes satisfied the requirements of ASTM A 305-53T.

A tension test was made of one coupon from each bar. The average yield point, ultimate strength, and percent elongation in 8 in. are listed in Table 5. The average modulus of elasticity was 29,000,000 psi.

The bars in each beam were matched so that bars of the same size would have approximately the same mechanical properties. The average yield points for reinforcement in each individual beam are given in Table 2. Typical stress-strain curves for intermediate and hard

TABLE 5—AVERAGE PROPERTIES OF REINFORCING BARS*

Bar size	Area, sq in.	Deformation, in.			Yield point, psi	Ultimate strength, psi	Elongation in 8 in., percent
		Spacing	Height	Gap			
Series N and Series A—Intermediate Grade							
# 9	0.989	0.714	0.067	0.191	44,700	78,300	22.3
# 7	0.595	0.571	0.050	0.160	46,600	80,500	20.6
# 3	0.111	0.250	0.020	0.045	51,200	76,200	20.8
# 3†					55,600	85,600	
Series H—Hard grade							
# 9	0.977	0.738	0.066	0.149	58,700	121,100	10.9
# 7	0.585	0.559	0.039	0.112	56,100	103,900	13.5
Series I—Intermediate grade							
# 9	0.983	0.741		0.161	44,000	80,500	20.3
# 7	0.597	0.561		0.134	57,800	80,500	19.3
# 4	0.191	0.333	0.032	0.057	45,300	65,800	15.6
# 3	0.115	0.261	0.022	0.067	46,100	73,700	18.6
# 2	0.051	—	—	—	54,300	77,200	12.1

*All reinforcing bars were made of billet steel meeting the requirements of ASTM A 15-54T. Except when noted otherwise, the deformations were perpendicular to the bar axis and met the requirements of ASTM A 305-53T.
One coupon was tested from each bar length. The deformation data were determined on five bars of each size selected at random.

†Bars used only for web reinforcement in Shear Spans 1 of Beams E6A2, C6A2, E3A2, E2A2, E2A3, and C2A2. They had deformations of the type shown in Fig. 3b.

grade bars are shown in Fig. 4, which includes data obtained by a mechanical extensometer and by SR-4 electric strain gages. The #7 hard grade bars gave stress-strain curves similar in shape to those of intermediate grade bars. The major difference was that in the former the initial plastic region showed a slight slope, while in the latter the initial plastic region was horizontal.

Fabrication and curing

All beams were cast in steel forms in an upright position. The bottom reinforcement was supported on steel chairs resting on the bottom of the forms. The top reinforcement was supported by the stirrups and, when needed, by a transverse spacer bar on the center line of the beam; where no stirrups were provided, it was suspended on wires fastened outside the beam. The stirrups were wired to the top and bottom longitudinal reinforcement; where no top reinforcement was provided, stirrups were supported by temporary longitudinal spacer bars which were removed before the forms were completely filled with concrete.

Concrete was placed in three batches. The first two batches were placed in the outer thirds of the beam, and the third batch was placed in the middle third. The concrete was compacted with an internal rod vibrator. Four to five 6 x 12-in. control cylinders were made from each batch. The cylinders were cast in steel forms and compacted with an internal rod vibrator. A few hours after casting the cylinders were capped with neat cement paste.

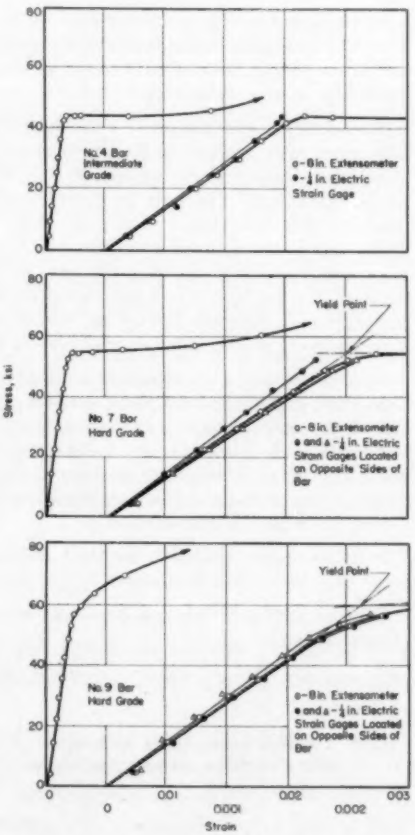


Fig. 4—Stress-strain curves for steel bars

On the day after casting forms were removed and the specimen and control cylinders were covered with wet burlap for 6 additional days of moist curing. Seven days after casting the burlap was removed and the specimen and control cylinders were stored in the laboratory and allowed to dry until tested at the age of approximately 28 days.

TEST METHOD

Equipment and Instrumentation

All continuous beams were tested in a 300,000-lb mechanical testing machine. The beam was supported on a screw jack and on a pedestal (Fig. 5), that formed the outside supports, and on a steel pipe dynamometer forming the center support. All three supports rested on the bed of the testing machine. The screw jack permitted vertical adjustments to correct for deformations of the center support and of the bed of the testing machine.

The center reaction was measured with the steel pipe dynamometer of 20,000-lb capacity, which was equipped with electric resistance SR-4 strain gages. Deflections along the center line of the beam's bottom surface were measured with six 0.001-in. dial indicators. These

were supported from a bridge attached to the beam over the end supports in beams of Series N and A, and to the end supports in beams of Series H and I as shown in Fig. 5. The changes in relative position of the three supports were checked with two additional dials attached to the bridge at the center support.

Steel strains were measured with Type A-7 electric resistance strain gages ¼ in. long. The gages were attached to the gap of the reinforcing bars through core holes formed in the sides of the specimens during casting. On the top longitudinal reinforcement, one gage was placed on each bar 2 in. from the center line of the center support. On the bottom reinforcement one gage was placed on each bar 2 in. inside from the point of maximum positive moment, and at the point of contraflexure. Corresponding gages on the two longitudinal bars were always placed in the opposite spans so that only one core hole was located at any cross section. Strains on the web reinforcement were measured only in Shear Spans 3. Four to ten gages were distributed along the lines of expected major diagonal tension cracks to obtain data for every stirrup location with respect to the center support.

Concrete strains were measured with Type A-1 electric resistance strain gages 13/16 in. long. One gage was placed along the center line of the bottom surface of the beam, on each side of the center support 2 in. from the center line of the support. In all beams in Series H and Series I one gage was placed along the center line of the top surface of the beam 2 in. inside the points of maximum positive moment. As all bearing blocks were 4 in. wide, half of the gage was under the bearing block; to accommodate the gage the blocks had a narrow groove along the bearing surface.

In a few beams, strains in the steel and concrete were also measured at several locations other than those described above.

Prior to testing all bearing blocks in contact with the concrete surface were bedded in plaster of Paris.

NOTE: 1. All Steel Bearing Blocks 4 × 6 × 1½"
2. All Steel Rollers and Balls 2" in Diameter

Series	Type of Loading	x	y
N and A	1/6 – Point	17.0"	34.0"
	1/3 – Point	34.0"	17.0"
	1/2 – Point	25.5"	25.5"
H and I	All Loadings	25.5"	25.5"

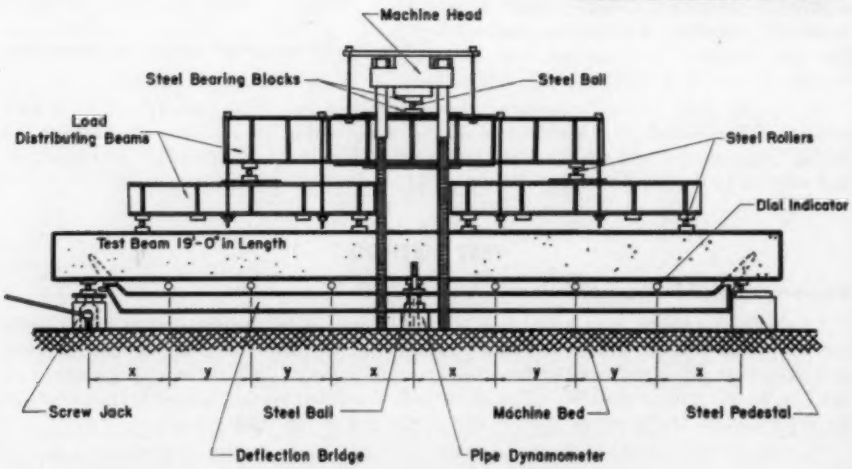


Fig. 5—Testing arrangement

Test procedure

Three different types of loading (Fig. 1) were used. The $\frac{1}{6}$-point loading consisted of four equal concentrated loads on the beam, applied at the outer sixth points of each span. The $\frac{1}{3}$-point loading consisted of four equal concentrated loads on the beam, applied at the third points of each span. The $\frac{1}{2}$-point loading consisted of one concentrated load at the center point of each span.

The load was applied through a system of distributing beams (Fig. 5). The arrangement in Fig. 5 corresponds to $\frac{1}{6}$-point loading. For $\frac{1}{3}$-point loading the inner bearing blocks were used, and for $\frac{1}{2}$-point loading the load was applied through the top distributing beam only.

The load was applied in increments up to failure of the specimen. Prior to application of the first load, steps were taken to make sure that the dead load of the beam was distributed to the supports in accordance with the elastic theory of beams on nondeflecting supports. During application of each increment of load the screw jack was frequently adjusted to correct for any changes in the relative position of the supports, thus simulating conditions existing in a beam supported on nondeflecting supports. After each increment, the load was stabilized and measurements were taken of the center reaction, deflections, steel strains, and concrete strains. In addition, the specimen was examined for crack formation, and a pictorial record was kept of changes in crack pattern. Testing of one specimen took from 4 to 6 hr.

The control cylinders were tested in a 300,000-lb hydraulic testing machine on the same day as the corresponding beam. In tests of cylinders from the third batch, strains were measured with an 8-in. extensometer.

TEST RESULTS

Test data compiled

The test data are summarized in Table 6, which includes loads at the formation of critical diagonal tension cracks, first yielding of reinforcement, and failure.

A diagonal tension crack is an inclined crack extending from the positive longitudinal reinforcement to the critical section, and making an angle of about 45 deg with the reinforcement. The diagonal tension cracking load is that load at which the diagonal tension crack has extended sufficiently into the beam to cause an appreciable redistribution of internal stresses. Diagonal tension cracking loads given in Table 6 were determined from visual observation and from a study of the load-steel strain curves and the load-deflection curves.

The load at first yielding of the reinforcement was determined from the load-strain curves for the reinforcement.

"Failure load" is the maximum load that the beam could carry. Many specimens continued to carry a load after reaching the maximum load. Whenever this was the case the test was continued until the load had dropped to approximately half the maximum. It is important to note, however, that should the maximum load have been maintained on the specimen for a short period, complete collapse would probably have occurred at that load.

Behavior before and at first diagonal tension cracking

In tests of all the beams three distinct stages were observed during loading: (1) beam in uncracked conditions, (2) beam cracked in tension only, and (3) beam with diagonal tension cracks in addition to flexural cracks. Further-

TABLE 6—DIAGONAL TENSION CRACKING, YIELD, AND FAILURE LOADS

Beam	Load P, kips, at first diagonal tension cracking				Load P, kips, at first yielding of reinforcement			Failure data			
	Shear Span				Longitudinal						
	3		2	1			Web, Shear Span 3	Load P, kips	Location*	Mode†	Center reaction, kips
	Top	Bottom			Top	Bottom					
Series N											
E6N1	59	130	150	80	—	—		160	3B	S	94.9
E6N2	52	100	—	70	—	—		123	3B	S	72.9
E6N3	52.5	120	—	80	—	—		140	3B	S	83.0
C6N1	50	128	—	90	—	—		160	3B	S	94.5§
C6N2	55.8	100	—	88.6	—	—		115	3B	S	68.2§
C6N3	60	99.3	—	100	—	—		140	3B	S	83.1§
E3N1	40	75	—	60	—	—		75	3	S‡	49.1
E3N2	38	70	—	—	—	—		70	3	S‡	45.9
C3N1	39	—	—	—	—	—		45	3	D–B	29.5
C3N2	40	—	—	—	—	—		71.8	3B	S–B	47.0
E2N1	38	38		—	—	—		38.5	3T	B	24.7
E2N2	34	34		—	—	—		39.8	3T	B	25.6
E2N3	30	—		—	—	—		33	3T	B	21.2§
C2N1	35	—		—	—	—		38	3T	B	24.4
C2N2	29.4	—		—	—	—		37.2	3T	B	23.9
Series A											
E6A1	48.5	160	160	74	—	—	—	190	2	D	114.6
E6A2	58.2	90	160	75.2	—	—	—	172.3	2	D	104.0
C6A1	50	145	—	120	—	—	183	198.5	3B	S	119.8
C6A2	58.3	135.3	180	100	—	—		194.7	2	D	117.3
E3A1	37	50	—	90	110	110	102	120	3B	F	79.7
E3A2	38	70	—	80	115	117	109	120	3B	F	79.7
C3A1	40	83	110	100	—	—	98	118	2	D	78.7
C3A2	40	100	—	91.7	—	—	108	119	3B	S	79.3
E2A1	30	38		—	—	80	70	85	3	S	60.0
E2A2	35	35		—	—	77	64	78	3	S	53.9
E2A3	30	40		80	—	—	67	84	3	S	57.9§
C2A1	30	40		—	—	—	—	65	3T	S	44.7
C2A2	30	40		70	—	—	55	80	3T	S	54.9§
Series H											
E6H1	64	130	—	90	—	220	—**	229.6	3B	S	136.9§
C6H1	56	170	—	70	—	—	—**	208.5	3B	S	123.8§
E6H2	48	110	—	80	—	—	—	208.5	3B	S	122.0§
C6H2	64	100	—	72	—	—	—	210.4	3B	S	123.6§
E3H1	36	65	—	65	—	120	—	144.5	3B	S	92.4§
C3H1	30	85	—	55	—	125	—	128.0	3B	S	81.4§
E3H2	36	80	—	60	—	—**	—	128.0	3B	S	82.5§
C3H2	30	—	—	60	—	—**	—	117.6	3B	S	74.3§
E2H1	30	35		75	—	90	100	103.7	3B	S–B	67.3§
C2H1	30	55		65	—	90	—	97.0	3B	S–B	65.8§
E2H2	30	55		55	—	85	85	89.6	3T	S–B	57.5§
C2H2	30	55		85	—	95	75	95.0	3T	S–B	63.7§
Series I											
E6I1	60	120	—	80	200	200	—	210.0	3B	F	121.7§
C6I1	50	140	—	80	195	205	—	205.0	3B	F	117.1§
E6I2	60	160	—	95	—	—	—	190.0	3B	S	110.5§
C6I2	50	140	—	80	—	195	—	195.0	3B	S	113.4§
E3I1	36	95	—	65	124	120	—	126.0	3B	F	81.2§
C3I1	35	60	—	60	120	120	—	122.0	2T	F	79.4§
E3I2	35	65	—	65	124	120	124	124.0	3B	F	80.1§
C3I2	36	95	—	65	120	115	120	123.0	1T	F	80.6§
E2I1	30	50		70	95	90	97	95.0	3T	F	67.1§
C2I1	30	40		70	95	95	104	105.0	1T	F	73.1§
E2I2	35	50		60	95	90	97	97.0	3T	F	67.9§
C2I2	30	50		50	95	90	85	95.0	1T	F	66.0§

*1T: top of Shear Span 1 adjacent to load bearing block; 2: in Shear Span 2; 2T: top of Shear Span 2 adjacent to outside load bearing block; 3: in Shear Span 3; 3B: bottom of Shear Span 3 adjacent to center support bearing block; 3T: top of Shear Span 3 adjacent to load bearing block.

†S = shear compression failure, D = diagonal tension failure, F = flexural failure, B = splitting bond failure.

‡Failed simultaneously with the formation of the second diagonal tension crack in Shear Span 3.

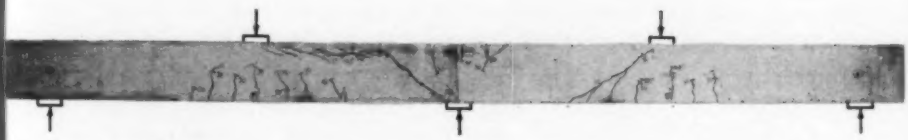
§Measured values; all other reactions are extrapolated quantities.

**The strain readings at failure indicated that the steel may have been on the verge of yielding.

Series N

E6N3 — Shear compression failure

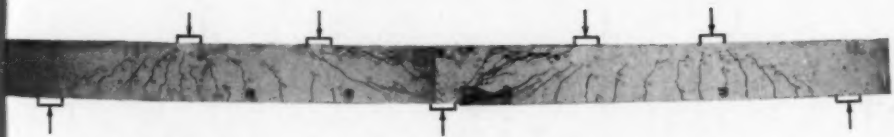
C3N2 — Shear compression–splitting bond failure



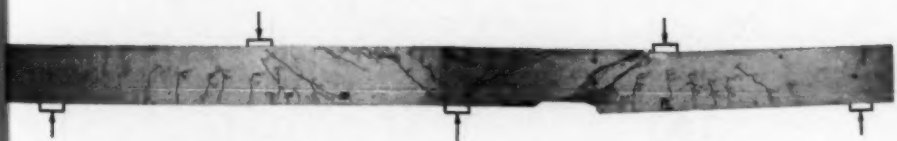
E2N1 — Splitting bond failure

Series A

E6A1 — Diagonal tension failure



E3A2 — Flexural failure



C2A1 — Shear compression failure

Fig. 6—Beams after failure. Examples from Series A and Series N

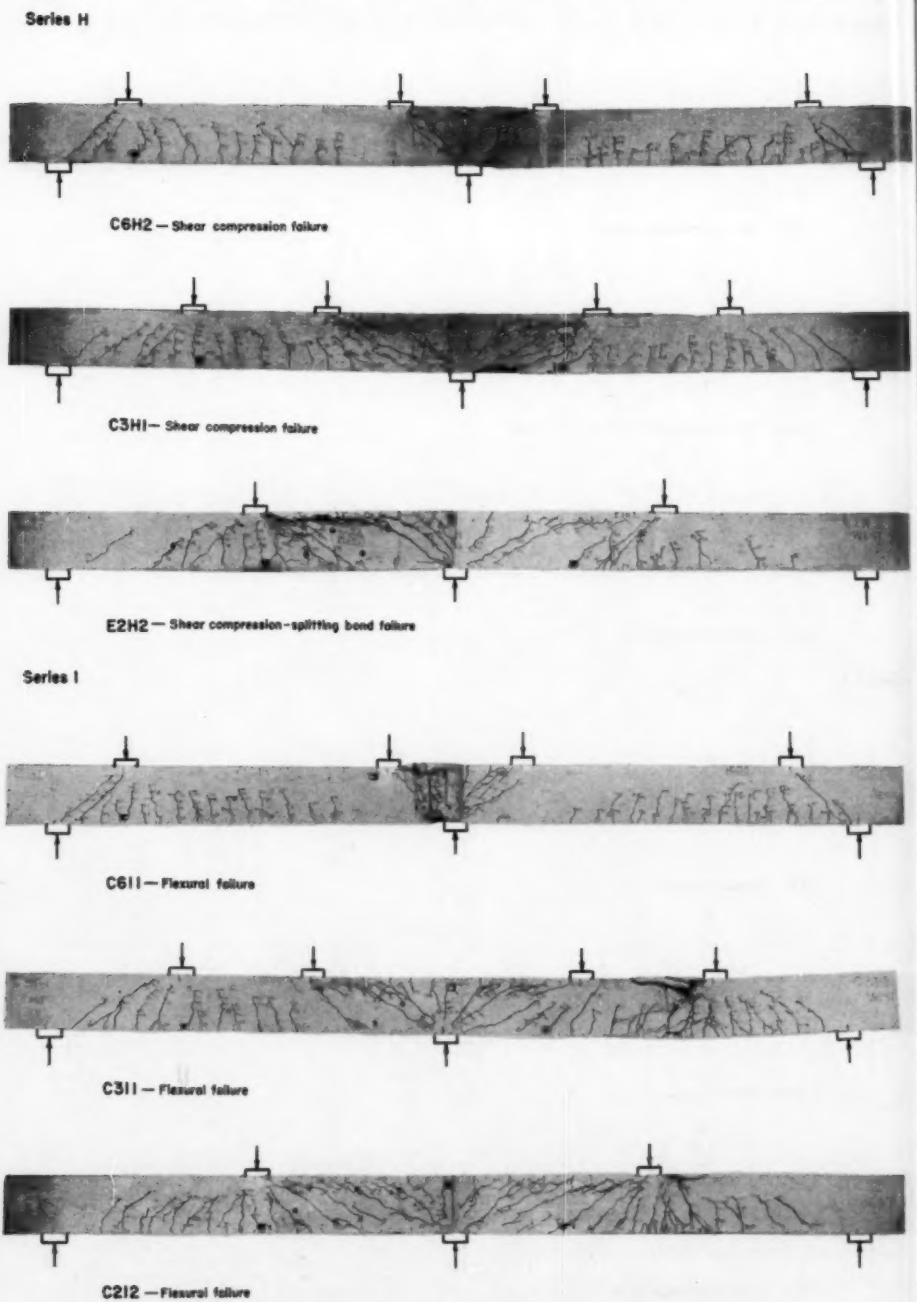

Series H
C6H2 — Shear compression failure
C3H1 — Shear compression failure
E2H2 — Shear compression-splitting bond failure
Series I
C6I1 — Flexural failure
C3I1 — Flexural failure
C2I2 — Flexural failure

Fig. 7—Beams after failure. Examples from Series H and Series I

more, at loads approaching failure, varying degrees of splitting along the longitudinal reinforcing bars and crushing at the sections of high compressive stresses were usually observed. The different types of cracks observed in the four series—flexural, diagonal tension, splitting, and crushing—are shown in Fig. 6 and 7.

Cracking had in general a profound influence on the over-all behavior of the beams. However, cracking had a small influence on the center reaction. Fig. 8 compares the experimental measurements of center reaction for various increments of loading for four typical beams with center reaction computed by elastic theory for a two-span continuous beam of uniform stiffness (full line) and with the reaction computed for two simple beams (dashed line).

Data in Fig. 8a, typical of beams of Series N and Series A, indicate that experimental values of the center reaction were almost identical to theoretical values computed by elastic theory. Deviations from the theoretical values were observed systematically only in beams without web reinforcement in Shear Span 3 at loads almost equal to the failure load. In those instances the center reaction value was always lower than the theoretical value for a two-span continuous beam; however, it was always substantially higher than half the total load.

The center reaction data of typical beams of Series H and Series I are shown in Fig. 8b. Although there was good agreement between theoretical elastic values and measured values for the center reaction, in no case were the two values identical. Deviations of up to 3.5 percent were observed, the measured reaction value being always smaller than the theoretical. It is noteworthy, however, that a small variation in the center reaction may change the moments in the member appreciably. For the beams under consideration computations show that a variation of 1 percent in the center reaction will change the maximum negative moment 4 percent and 6 percent for ½-point loading and the ⅙-point loading, respectively.

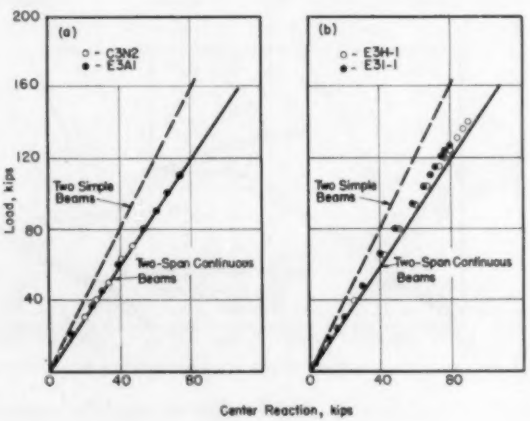


Fig. 8—Reaction of center support

Typical load-deformation curves are shown in Fig. 9. It will be noted that after diagonal tension cracking, which occurred in beams with ⅓-point loading at a load of about 40 kips, both the rate and magnitude of measured deflections exceeded the computed values. Except at low loads and after yielding, measured steel strains agreed well with the computed values for cracked sections.

Diagonal tension cracking loads in Shear Span 3 are recorded in Table 6. It was observed in this investigation that the cracks developed in three ways: (1) diagonal cracks started at the level of top reinforcement and extended toward the center support; (2) diagonal crack formed at about middepth of the beam and extended upward and downward simultaneously; and (3) diagonal tension crack started at the tip of slightly inclined flexural cracks, connecting a number of them.

Diagonal tension cracking loads listed in Table 6 under the heading "load at first diagonal tension cracking in shear Span 3, top" were the loads at which the first diagonal tension crack formed in every beam. The cracks formed progressively so that the loads corresponding to their formation were

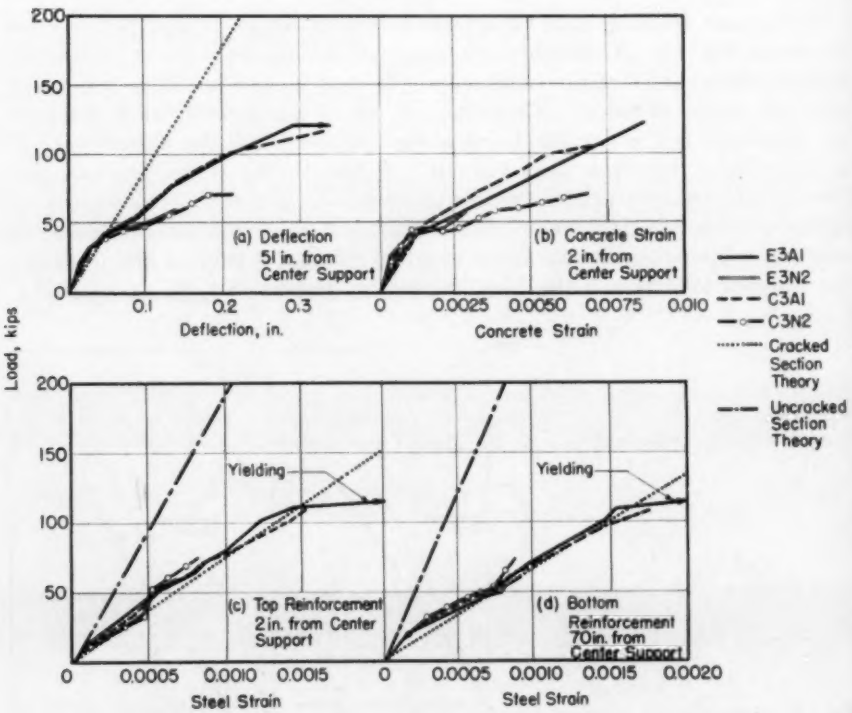


Fig. 9—Typical load-deformation curves

difficult to determine. Their growth was slower in beams with web reinforcement than in beams without web reinforcement, and in beams with $\frac{1}{6}$-point loading than in beams with either $\frac{1}{3}$- or $\frac{1}{2}$-point loading.

Behavior after formation of first diagonal tension crack

After formation of the first diagonal tension crack, all beams were capable of sustaining further increase in load. The load increase was accompanied by an increase in the amount and extent of flexural and diagonal tension cracking and by a redistribution of internal stresses. The redistribution of stresses was particularly well indicated by the steel strain measurements taken at points of contraflexure as shown in Fig. 10, and by concrete strain

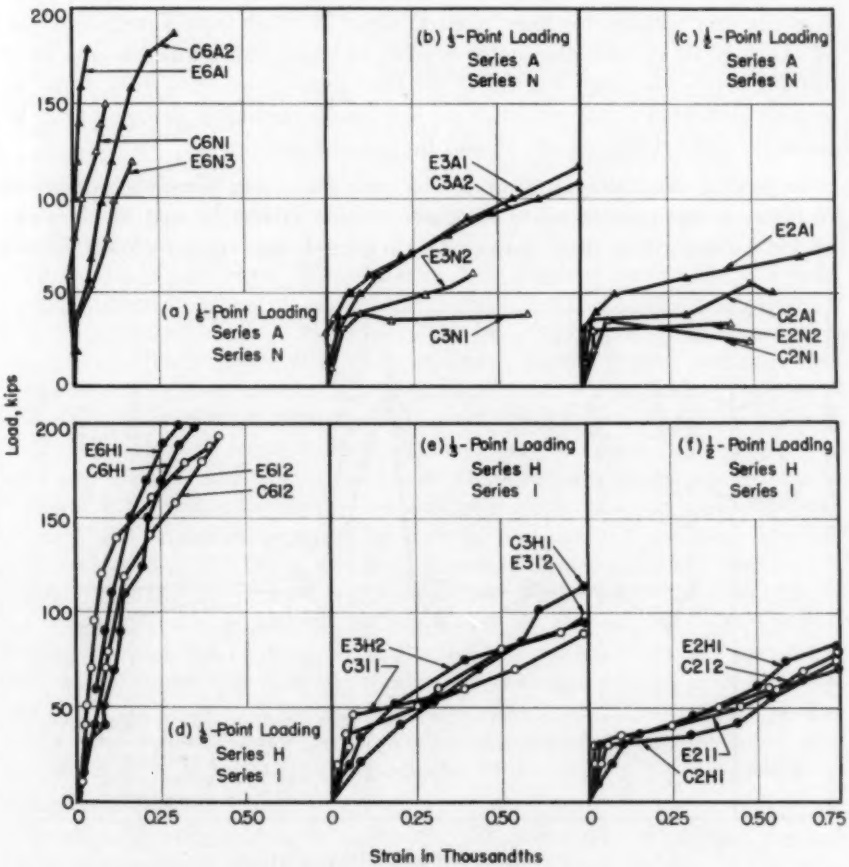


Fig. 10—Bottom steel strains at section of theoretical zero moment

measurements taken on the bottom surface near the center support as shown in Fig. 11.

Strains at the theoretical point of contraflexure were measured in both bottom and top longitudinal reinforcement and are shown in Fig. 10 for the bottom reinforcement in typical beams. For ⅓-point and ½-point loadings these strains were rather small up to approximately the first diagonal tension cracking load; upon further increments of load the steel strains increased appreciably. The ⅙-point loading showed a gradual increase in steel strains with increases in load; measurable strains were usually observed at loads varying between 40 and 120 kips.

Distribution of concrete strains along the bottom surface of the beam was measured in three beams as shown in Fig. 11. The points of contraflexure were located 20.5, 25.5, and 28 in. from the center support, for the ⅙-point, ⅓-point, and ½-point loadings, respectively. The high tensile strain recorded by the gage 10 in. from the center support in beam E6N1 was due to a crack forming inside the gage length. Note that gages more than 4 in. from the support indicated either no strain or low tensile strains in a region that according to the elastic theory should be in compression.

In general crack development in the two spans was almost symmetrical. In some beams corresponding diagonal tension cracks formed at the same load in corresponding shear spans, while in other beams cracking loads differed

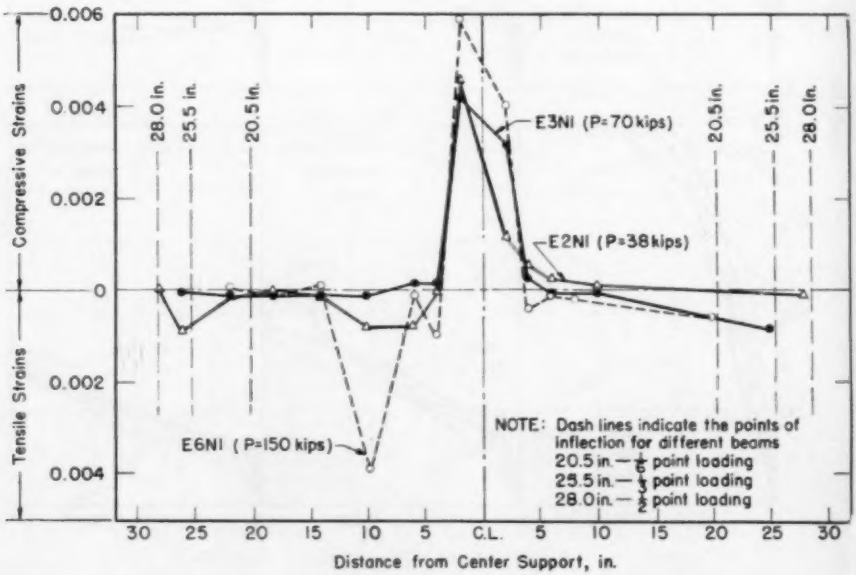


Fig. 11—Distribution of concrete strains on bottom surface along the beams

up to 30 percent. The smaller of the two corresponding diagonal tension cracking loads is listed in Table 6.

After formation of the first diagonal tension crack in Shear Span 3, additional diagonal cracks developed at some or all of the following locations: in the adjacent Shear Span 3, at the bottom of Shear Spans 3, in Shear Spans 1, and in Shear Spans 2.

Diagonal tension cracks in Shear Spans 1 extended from the level of bottom longitudinal steel toward the first load point. Crack formation was gradual, which is believed to be partly responsible for the scatter of cracking load in Shear Spans 1 for specimens of the same type, as shown in Table 6. At no time did these cracks open up widely. In several beams with ⅓- and ½-point loading no diagonal tension cracks formed in Shear Spans 1, evidently because the beams failed at a load lower than that necessary for the formation of that crack.

The second diagonal crack in Shear Spans 3 extended from the level of bottom reinforcement toward the loading block. Loads causing the second diagonal crack are listed in Table 6 under the heading "load at first diagonal tension cracking in Shear Span 3, Bottom." These cracks were observed in all beams with web reinforcement of ½-point and ⅓-point loading, and in most beams subjected to ⅙-point loading. In addition to the first and the second diagonal cracks, a number of minor diagonal cracks formed in Shear Spans 3 of several beams. In most cases, however, these cracks were not well defined or continuous.

In five beams in Series N and Series A a diagonal tension crack formed in Shear Span 2, where in accordance with ACI 318-51, web reinforcement was not required. Four of them failed suddenly and violently at loads either equal to or slightly higher than those at which the cracks were first observed. These were the most violent of all failures. None of the beams of Series H and Series I developed well-defined diagonal tension cracks in Shear Span 2. Some indications of initial inclined cracking were observed, but it is possible that web reinforcement prevented full development of diagonal tension cracks.

Beam C3I2 failed in flexure and collapsed under unusual circumstances. A fully plastic condition (yielding of both top and bottom reinforcement) had developed in the beam, and the amount of deformation had been appreciable. The load started dropping slowly and the drop-off was accompanied by concrete crushing next to the inner face of the outside loading block. Suddenly, when the load was approximately 10 percent below the maximum, the beam collapsed by diagonal tension. The diagonal crack extended from the top of the beam in Shear Span 2, next to the loading block, to the bottom of the beam. It is believed that high compressive stresses in combination with a state of high shear precipitated collapse of the beam.

Behavior at failure

Four modes of failure were observed in these tests: diagonal tension, flexural, shear compression, and splitting bond failures. Table 6 lists the load at failure, location of the failure section, and the mode of failure for each beam.

Series N

All beams without web reinforcement failed in Shear Span 3, mode of failure varying with type of loading. After the maximum load was reached there was either a substantial drop in load carrying capacity, or a complete collapse of the beam.

All beams with $\frac{1}{6}$-point loading failed in shear compression by crushing of concrete adjacent to the center support bearing block. Concrete strains were high at the section of failure, indicating a large concentration of stresses (Table 7).

Beams with $\frac{1}{3}$-point loading showed basically two types of failure, in which the length of negative longitudinal reinforcement appears to have had an effect. Beams with extended top steel failed simultaneously with the formation of the second diagonal tension crack in Shear Span 3. Beams with cutoff top reinforcement failed along an existing diagonal tension crack, with splitting taking place along the top longitudinal reinforcement, and extending to within a few inches of the cutoff points (Beam C3N2 in Fig. 6). Strain measurements after failure indicated the possibility that final failure was caused by slippage of the top bars. The slippage was probably a consequence of the preceding splitting which, in turn, had its origin in the formation of the first diagonal tension crack.

Beams with $\frac{1}{2}$-point loading failed by splitting bond failures along the top longitudinal reinforcement at loads only slightly in excess of the first diagonal tension cracking loads. In beams with extended top reinforcement the splitting extended to the load points (Beam E2N1 in Fig. 6), whereas in beams with cutoff top reinforcement splitting extended to the cutoff points.

Series A

Beams with ACI Code web reinforcement failed either in Shear Span 2 or in Shear Span 3. Failures which occurred in Shear Span 2 were sudden and complete. Failures in Shear Span 3 were usually gradual and, with one exception, occurred after yielding of web reinforcement (Table 6), followed in some cases by yielding of the longitudinal reinforcement. None of the beams designed according to ACI 318-51 reached full flexural capacity.

Beams with $\frac{1}{6}$-point loading failed either in Shear Span 2 or Shear Span 3. Failures in Shear Span 2 were characteristic diagonal tension failures although they did not occur simultaneously with the formation of the diagonal tension crack (Beam E6A1 in Fig. 6). One beam failed in Shear Span 3 by shear compression with concrete crushing adjacent to the center support after yielding of web reinforcement.

Beams with $\frac{1}{3}$-point loading failed either in Shear Span 2 or Shear Span 3 after yielding of web reinforcement in Shear Span 3. Failure in Shear Span 2 occurred by diagonal tension. Flexural and shear compression failures occurred in Shear Span 3 by crushing of concrete adjacent to the center support.

TABLE 7—MAXIMUM DEFORMATION AT FAILURES*

Beam	Deflections at midspan, in.	Concrete strains	Strains in reinforcement		
			Top	Bottom	Web†
Series N					
E6N1	0.160	0.00660	0.00100	0.00125	
E6N2	0.146	0.00550	0.00092	0.00100	
E6N3	0.140	0.00500	0.00112	0.00120	
C6N1	0.180	0.00950	0.00122	0.00127	
C6N2	0.180	0.00900	0.00091	0.00076	
C6N3	0.174	0.00600	0.00102	0.00115	
E3N1	0.170	0.00490	0.00080	0.00102	
E3N2	0.164	0.00375	0.00086	0.00100	
C3N1	0.060	0.00160	0.00055	0.00050	
C3N2	0.185	0.00625	0.00078	0.00090	
E2N1	0.110	0.00390	0.00046	0.00068	
E2N2	0.130	0.00225	0.00045	0.00054	
E2N3	0.052	0.00085	0.00064	0.00080	
C2N1	0.076	0.00184	0.00038	0.00076	
C2N2	0.062	0.00064	0.00054	0.00056	
Series A					
E6A1	0.180	0.00700	0.00160	0.00172	0.00146
E6A2	0.142	0.00625	0.00128	0.00132	0.00146
C6A1	0.220	0.00560	0.00163	0.00177	0.00218‡
C6A2	0.230	0.00575	0.00172	0.00167	0.00173
E3A1	0.331	0.00835	0.01044‡	0.01044‡	0.00269‡
E3A2	0.350	0.00750	0.00210‡	0.00737‡	0.00398‡
C3A1	0.440	0.00850	0.00170	0.00156	0.00300‡
C3A2	0.315	0.00875	0.00143	0.00190	0.00224‡
E2A1	0.344	0.00600	0.00113	0.00653‡	0.00401‡
E2A2	0.272	0.00565	0.00110	0.00266‡	0.00725‡
E2A3	0.290	0.00605	0.00120	0.00184	0.00712‡
C2A1	0.190	0.00200	0.00115	0.00100	0.00150
C2A2	0.220	0.00375	0.00100	0.00166	0.00218‡
Series H					
E6H1	0.346	0.00970	0.00170	.....‡	0.00155
C6H1	0.262	0.00996	0.00187	0.00186	0.00846‡
E6H2	0.279	0.00955	0.00125	0.00195	0.00133
C6H2	0.277	0.00910	0.00144	0.00194	0.00145
E3H1	0.534	0.00703	0.00206	0.00870‡	0.00170
C3H1	0.480	0.00900	0.00153	0.00203‡	0.00137
E3H2	0.320	0.00578	0.00146	0.00204	0.00134
C3H2	0.363	0.00716	0.00154	0.00213	0.00091
E2H1	0.636	0.00702	0.00191	0.00700†	0.01318‡
C2H1	0.420	0.00372	0.00134	0.00948‡	0.00193
E2H2	0.400	0.00705	0.00116	0.00260‡	0.00516‡
C2H2	0.322	0.00512	0.00142	0.00461‡	0.00636‡
Series I					
E6I1	0.329	0.00792	0.00732‡	0.01250‡	0.00175
C6I1	0.290	0.01233	0.00900‡	0.01340‡	0.00165
E6I2	0.259	0.00694	0.00150	0.00180	0.00130
C6I1	0.278	0.01279	0.00197	0.00608‡	0.00181
E3I1	0.835	0.00813	0.00815‡	0.01244‡	0.00134
C3I1	0.802§	0.01104	0.01141‡	0.01560‡	0.00144
E3I2	0.453§	0.00832	0.00540‡	0.01457‡	0.01212‡
C3I2	0.567§	0.00898	0.01156‡	0.01976‡	0.00303‡
E2I1	0.685§	0.00673	0.01354‡	0.01243‡	0.00189
C2I1	1.225§	0.00978	0.01501‡	0.01814‡	0.00285‡
E2I2	0.712§	0.00884	0.01141‡	0.01253‡	0.00250‡
C2I2	0.815§	0.00733	0.00877‡	0.01414‡	0.01415‡

*With a few exceptions the values listed are extrapolated quantities. Deflections were measured at midspan, concrete strains at the center line of the bottom beam surface 2 in. from the center support, top steel strains 2 in. from the center support, bottom steel strains at the critical section in Shear Span 1, and web steel strains on the stirrups in Shear Span 3.

†In Shear Span 3.

‡Yielding observed before failure load was reached.

§Deflections increased beyond value given while load was decreasing.

All beams with ½-point loading failed by shear compression in Shear Span 3. Final failure was either by concrete crushing adjacent to the center support or by concrete crushing adjacent to the loading block. In two cases yielding of the bottom longitudinal reinforcement was observed.

In summary, the amount of web reinforcement used in Shear Span 3 was inadequate to prevent shear failures. The stirrups always yielded at loads lower than those required for yielding of longitudinal reinforcement. Usually the shear failure occurred before yielding of the longitudinal steel. Furthermore, omission of web reinforcement in Shear Span 2 for beams with ⅙-point and ⅓-point loadings was unwarranted. Also, it seemed that cutting off the longitudinal reinforcement tended to aggravate the critical situation introduced by the inadequate amount and spacing of web reinforcement.

Series H

All beams with hard grade longitudinal reinforcement failed by shear compression in Shear Span 3. Failures were sudden for beams with ⅙-point loading and gradual for beams with ⅓-point and ½-point loadings. In beams in which a diagonal crack crossed a strain gage, substantial yielding of web reinforcement was detected. In the remaining beams strains in the stirrups were large at failure but below the yield point value. Nevertheless, in all beams failing in shear compression the main diagonal cracks widened noticeably before failure. It seems logical to assume then that there was some yielding of web reinforcement in every case.

The failure of beams with ⅙-point loading was actually a combination of crushing of concrete next to the center support and shearing off of one part of the beam with respect to the other, with heavy spalling of concrete exposing the stirrups completely as shown in Fig. 7 for beam C6H2. A comparison of the results seems to indicate that beyond a certain value any increase in the percentage of vertical web reinforcement adds little to the shear capacity of beams.

Failure of beams with ⅓-point loading was by gradual crushing of concrete next to the center support. In two beams strain measurements indicated yielding of longitudinal steel.

Failure of beams with ½-point loading was accompanied by splitting of concrete along the top reinforcement with definite yielding of both bottom longitudinal steel and web reinforcement. Crushing of concrete next to the center support occurred in beams with extended top reinforcement. In beams with cutoff top steel, concrete adjacent to the loading block crushed at failure, with only slight signs of crushing occurring next to the center support. However, it is believed that the failure of these beams was precipitated by splitting of concrete along the top steel (E2H2 in Fig. 7).

Series I

Beams with intermediate grade longitudinal reinforcement and an amount of web reinforcement larger than that required by ACI 318-51 failed in flexure

in all cases except two. Yielding of the bottom reinforcement always preceded yielding of the top reinforcement. In half the specimens yielding of web reinforcement was observed at failure or at loads close to failure. It is possible that deformations resulting from yielding of the longitudinal steel may have precipitated yielding of the web reinforcement. This aspect merits further study.

In some beams it was possible to observe that any increase in load after yielding of the longitudinal steel in the span was resisted by the center reaction alone, clearly indicating the formation of fully developed plastic hinges in both spans.

Beams with $\frac{1}{6}$-point loading failed either in flexure or in shear compression. Beams failing in flexure had no apparent yielding of web reinforcement. Final failure was by crushing of concrete next to the center support (C6I1 in Fig. 7). Note that beams that failed in shear had a larger percentage of web reinforcement than the two beams failing in flexure.

Failure of beams with $\frac{1}{3}$-point loading was preceded by large deflections. Beams with extended reinforcement failed by crushing of the compression zone next to the center support, with concrete splitting along the top reinforcement. Beams with cutoff steel failed by crushing of concrete in the outer face of Shear Span 2 (C3I1 in Fig. 7). It appears that the absence of compression steel at the outside end of Shear Span 2 was a factor favoring failure at that point.

Beams with $\frac{1}{2}$-point loading failed after extensive yielding of longitudinal steel and web reinforcement. In beams with extended steel final failure was caused by concrete crushing on the outer face of Shear Span 3, accompanied by splitting of the concrete along the top steel. In beams with cutoff reinforcement final failure was by crushing of concrete in Shear Span 1 adjacent to the load point (C2I2 in Fig. 7).

ANALYSIS OF TEST RESULTS

Nominal shearing stress at diagonal tension cracking

For all beams tested, the center reaction was always in reasonably good agreement with values computed by the elastic theory of structures. The nominal shearing stress at diagonal tension cracking load may be evaluated as

$$v_c = \frac{8V_c}{7bd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The value V_c in each shear span may be computed from the cracking loads listed in Table 6.

The test values of v_c evaluated from Eq. (1) are listed in Table 8 as "v_c test." As the table indicates, diagonal tension cracks in Shear Spans 1 and 3 of beams subjected to $\frac{1}{6}$-point loading formed at the highest values of nominal

shearing stress. Diagonal tension cracks in Shear Span 2 for all beams formed at the lowest values of nominal shearing stress. This is in agreement with earlier studies[10] which have shown that the nominal shearing stress at first diagonal tension cracking is a function of the quantity

$$\frac{(M/V)_c}{npd}$$

where $(M/V)_c$ is the ratio of the moment to shear at the critical section of a diagonal tension crack. For the beams tested $(M/V)_c$ is equal to a_c; values of a_c are listed in Table 8. It may be seen that a_c is largest for Shear Spans 2 and smallest for Shear Spans 1 and 3 with all types of loading.

The calculated value for the nominal shearing stress was computed from the equation

$$v_c = \left[1.425 + \frac{6 + 39\,b/d}{\dfrac{(M/V)_c}{npd} + 10}\right]\sqrt{f_c'} \qquad (2)$$

advanced by Bower[11] as a modification of an equation formulated by Morrow and Viest.[10] The quantity $(M/V)_c = a_c$ is governed by the following relations[10]

$$a_c = 0 \text{ for } d \geqq a$$

$$a_c = a - d \text{ for } a \geqq d \geqq \frac{a}{2}$$

$$a_c = \frac{a}{2} \text{ for } \frac{a}{2} \geqq d$$

Eq. (2) was developed from an investigation of simply supported reinforced concrete beams with one or two overhangs. It was found to yield values in good agreement with test values for this investigation, as may be seen in Table 8 and Fig. 12. Fig. 12 includes only those beams that had no web reinforcement in one or more spans. It should be noted that the best agreement between computed and test values occurs in Shear Spans 2 and 3. This seems reasonable, considering that Shear Spans 1 had a higher percentage of web reinforcement which prevented a faster spreading of the diagonal crack. Furthermore, the cracks in Shear Span 1 were detected only by visual observation. Shear Spans 3 had also a high percentage of web reinforcement. However, readings of strain gages attached to the stirrups aided in determining the load at which the diagonal cracks started to develop. Diagonal cracks in Shear Spans 2 were detected by visual observations only. The percentage of web reinforcement in this span was either small or zero, so that the diagonal crack could be readily observed.

TABLE 8—NOMINAL SHEARING STRESS AT DIAGONAL TENSION CRACK

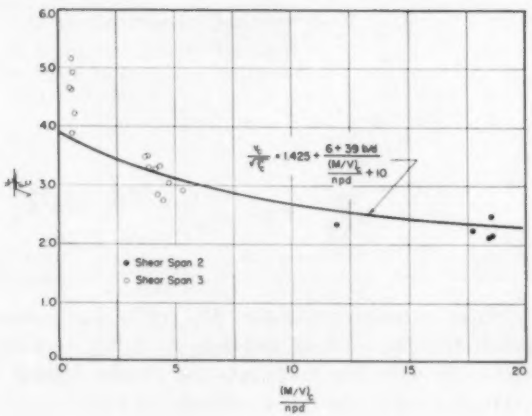
Beam	Shear Span 1			Shear Span 2			Shear Span 3		
	v_e test, psi	v_c calc., psi	$\frac{v_e \text{ test}}{v_c \text{ calc.}}$	v_e test, psi	v_c calc., psi	$\frac{v_e \text{ test}}{v_c \text{ calc.}}$	v_e test, psi	v_c calc., psi	$\frac{v_e \text{ test}}{v_c \text{ calc.}}$
	a = 15.0 in.		a_c = 2.5 in.	a = 62.6 in.		a_c = 31.3 in.	a = 15.6 in.		a_c = 3.1 in.
E6N1	239	210	1.14	118	130	0.91	272	215	1.26
E6N2	205	184	1.11	—	—	—	237	188	1.26
E6N3	234	203	1.15	—	—	—	239	208	1.15
C6N1	261	214	1.22	—	—	—	223	212	1.05
C6N2	259	194	1.34	—	—	—	257	183	1.41
C6N3	293	199	1.47	—	—	—	276	206	1.34
E6A1	221	202	1.09	125	134	0.93	223	218	1.02
E6A2	222	201	1.10	124	130	0.95	262	200	1.31
C6A1	360	208	1.73	—	—	—	230	204	1.13
C6A2	297	203	1.46	141	131	1.08	267	212	1.26
E6H1	272	207	1.31	—	—	—	291	218	1.34
E6H2	257	229	1.12	—	—	—	209	221	0.95
C6H1	210	208	1.01	—	—	—	248	213	1.16
C6H2	224	221	1.01	—	—	—	291	215	1.35
E6I1	253	220	1.15	—	—	—	265	223	1.19
E6I2	295	208	1.42	—	—	—	270	213	1.27
C6I1	262	226	1.16	—	—	—	218	218	1.00
C6I2	255	201	1.27	—	—	—	220	200	1.10
	Average		1.24	Average		0.97	Average		1.18
	Standard deviation		0.183				Standard deviation		0.132
	a = 32.0 in.		a_c = 16.0 in.	a = 40.5 in.		a_c = 20.3 in.	a = 23.5 in.		a_c = 11.0 in.
E3N1	150	161	0.93	—	—	—	203	198	1.03
E3N2	—	—	—	—	—	—	193	188	1.03
C3N1	—	—	—	—	—	—	198	185	1.07
C3N2	—	—	—	—	—	—	201	185	1.09
E3A1	229	156	1.47	—	—	—	189	191	0.99
E3A2	199	152	1.31	—	—	—	189	182	1.04
C3A1	249	158	1.58	136	149	0.91	199	183	1.09
C3A2	224	155	1.44	—	—	—	201	181	1.11
E3H1	173	157	1.10	—	—	—	185	203	0.91
E3H2	166	173	0.96	—	—	—	170	190	0.90
C3H1	148	158	0.94	—	—	—	148	175	0.85
C3H2	164	158	1.04	—	—	—	148	175	0.85
E3I1	173	169	1.02	—	—	—	172	200	0.86
E3I2	176	166	1.06	—	—	—	175	186	0.94
C3I1	157	163	0.96	—	—	—	174	200	0.87
C3I2	173	151	1.15	—	—	—	179	182	0.98
	Average		1.15	Average		0.91	Average		0.98
	Standard deviation		0.215	Standard deviation		0.000	Standard deviation		0.091
	a = 49.0 in.		a_c = 24.5 in.				a = 25.8 in.		a_c = 12.9 in.
E2N1	—	—	—				193	201	0.96
E2N2	—	—	—				178	167	1.07
E2N3	—	—	—				152	175	0.87
C2N1	—	—	—				180	185	0.97
C2N2	—	—	—				154	172	0.90
E2A1	—	—	—				157	186	0.84
E2A2	—	—	—				183	166	1.10
E2A3	184	133	1.38				155	173	0.90
C2A1	—	—	—				156	180	0.87
C2A2	160	138	1.16				156	174	0.90
E2H1	181	141	1.28				150	171	0.88
E2H2	136	139	0.98				157	173	0.91
C2H1	158	139	1.14				155	177	0.88
C2H2	204	152	1.34				141	185	0.76
E2I1	172	149	1.15				148	182	0.82
E2I2	149	146	1.02				180	184	0.98
C2I1	172	155	1.12				158	194	0.82
C2I2	120	133	0.90				160	166	0.96
	Average		1.15				Average		0.91
	Standard deviation		0.147				Standard deviation		0.083
	Over-all average		1.19	Over-all average		0.96	Over-all average		1.02
	Over-all standard deviation		0.192	Over-all standard deviation		0.064	Over-all standard deviation		0.163

TABLE 9—NOMINAL SHEARING STRESS AT FAILURE LOADS

Beam	Shear Span 1		Shear Span 2		Shear Span 3	
	v_u, psi	$\frac{v_u}{f_c'}$	v_u, psi	$\frac{v_u}{f_c'}$	v_u, psi	$\frac{v_u}{f_c'}$
	$r = 0.0092$, $a/d = 1.20$		$r = 0$, $a/d = 5.01$		$r = 0$, $a/d = 1.25$	
E6N1	492	0.159	112	0.036	*723**	0.209
E6N2	370	0.140	84	0.032	*551*	0.212
E6N3	422	0.128	96	0.029	*626*	0.193
C6N1	478	0.126	107	0.028	*698*	0.209
C6N2	346	0.108	78	0.024	*520*	0.211
C6N3	422	0.134	96	0.030	*634*	0.201
	$r = 0.0092$, $a/d = 1.20$		$r = 0$, $a/d = 5.01$		$r = 0.0133$, $a/d = 1.25$	
E6A1	569	0.168	*148**	0.044	873	0.245
E6A1	510	0.159	*131*	0.041	776	0.260
C6A1	598	0.172	155	0.045	*913*	0.295
C6A2	581	0.183	*149*	0.047	890	0.263
	$r = 0.0092$, $a/d = 1.20$		$r = 0.00039$, $a/d = 5.01$		$r = 0.0222$, $a/d = 1.25$	
E6H1	787	0.230	162	0.047	*1060*	0.303
C6H1	626	0.182	145	0.042	*920*	0.265
	$r = 0.0092$, $a/d = 1.20$		$r = 0.00039$, $a/d = 5.01$		$r = 0.0167$, $a/d = 1.25$	
E6H2	656	0.157	135	0.032	*935*	0.255
C6H2	661	0.172	140	0.036	*955*	0.280
	$r = 0.0092$, $a/d = 1.20$		$r = 0.00039$, $a/d = 5.01$		$r = 0.0178$, $a/d = 1.25$	
E6I1	657	0.170	121	0.031	940†	0.256
C6I1	664	0.165	126	0.031	900†	0.256
	$r = 0.0092$, $a/d = 1.20$		$r = 0.00039$, $a/d = 5.01$		$r = 0.0222$, $a/d = 1.25$	
E6I2	594	0.174	116	0.034	*864*	0.261
C6I2	625	0.202	122	0.039	*864*	0.292
	$r = 0.0061$, $a/d = 2.56$		$r = 0$, $a/d = 5.29$		$r = 0$, $a/d = 1.88$	
E3N1	194	0.053	87	0.024	*375*	0.096
E3N2	181	0.054	81	0.024	*350*	0.102
C3N1	117	0.035	52	0.016	*226*	0.069
C3N2	185	0.056	84	0.026	*355*	0.107
	$r = 0.0061$, $a/d = 2.56$		$r = 0$, $a/d = 5.29$		$r = 0.0061$, $a/d = 1.88$	
E3A1	308	0.094	149	0.045	611†	0.167
E3A2	301	0.095	146	0.046	595†	0.085
C3A1	294	0.084	*146*	0.042	588	0.180
C3A2	291	0.085	143	0.042	*598*	0.190
	$r = 0.0061$, $a/d = 2.56$		$r = 0.0017$, $a/d = 5.29$		$r = 0.0111$, $a/d = 1.88$	
E3H1	396	0.116	164	0.048	*734*	0.185
C3H1	355	0.103	132	0.038	*622*	0.212
	$r = 0.0061$, $a/d = 2.56$		$r = 0.0017$, $a/d = 5.29$		$r = 0.0089$, $a/d = 1.88$	
E3H2	355	0.085	144	0.035	*613*	0.169
C3H2	325	0.093	116	0.033	*571*	0.196
	$r = 0.0061$, $a/d = 2.56$		$r = 0.0017$, $a/d = 5.29$		$r = 0.0095$, $a/d = 1.88$	
E3I1	328	0.079	133	0.032	635†	0.166
C3I1	315	0.084	136	0.037	616†	0.159
	$r = 0.0061$, $a/d = 2.56$		$r = 0.0017$, $a/d = 5.29$		$r = 0.0111$, $a/d = 1.88$	
E3I2	338	0.090	139	0.037	624†	0.197
C3I2	325	0.109	147	0.049	620†	0.199
	$r = 0.0061$, $a/d = 3.92$				$r = 0$, $a/d = 2.06$	
E2N1	98	0.023			196§	0.045
E2N2	106	0.035			195§	0.068
E2N3	88	0.026			156§	0.050
C2N1	99	0.026			182§	0.051
C2N2	101	0.037			183§	0.062
	$r = 0.0061$, $a/d = 3.92$				$r = 0.0037$, $a/d = 2.06$	
E2A1	190	0.050			*457*	0.127
E2A2	182	0.064			*411*	0.151
E2A3	193	0.066			*456*	0.152
C2A1	153	0.047			*338*	0.102
C2A2	184	0.056			*414*	0.134
	$r = 0.0061$, $a/d = 3.92$				$r = 0.0082$, $a/d = 2.06$	
E2H1	274	0.081			*500‡*	0.157
C2H1	238	0.073			*512‡*	0.163
	$r = 0.0061$, $a/d = 3.92$				$r = 0.0052$, $a/d = 2.06$	
E2H2	236	0.071			*451‡*	0.153
C2H2	223	0.052			*474‡*	0.131
	$r = 0.0061$, $a/d = 3.92$				$r = 0.0082$, $a/d = 2.06$	
E2I1	208	0.055			515†	0.154
C2I1	242	0.058			566†	0.149
	$r = 0.0061$, $a/d = 3.92$				$r = 0.0067$, $a/d = 2.06$	
E2I2	218	0.061			524†	0.155
C2I2	218	0.075			519†	0.198

*The italicized values represent nominal shearing stresses (computed as $v_u = 8V_u/7bd$) in the shear span in which failure occurred by either shear compression or by diagonal tension. For modes and locations of failure see Table 6.
†Flexural failure.
‡Shear compression failure accompanied by splitting along reinforcement.
§Bond failures.

Fig. 12—Diagonal tension cracking



Nominal shearing stress at shear compression failure

Nominal shearing stresses at failure were computed for all shear spans from the experimental failure loads and the center reaction values listed in Table 6 as

$$v_u = \frac{8V_u}{7bd}$$

These values of v_u are listed in Table 9. The values in italic type represent the stresses in those shear spans in which either shear or diagonal tension failures occurred. The italicized values vary from 131 to 1060 psi, and the corresponding values of v_u/f_c' vary from 0.041 to 0.303.

Moment at shear compression failure

In recent studies of reinforced concrete beams failing in shear several investigators have postulated the hypothesis that a shear compression failure occurs when the moment at the critical section reaches a certain limiting value.[3,7,8] If this assumption is correct, equations for the limiting shear moment capacity of prismatic beams presented in Reference 7 should be applicable also to continuous beams.

Shear moment capacities of beams without web reinforcement in Shear Span 3 were computed from Eq. (3a) as given in Reference 7,

$$\frac{M_s'}{bd^2} = Npf_s\left[Z - k_2\frac{Npf_s}{k_1k_3f_c'}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where

$$N = 1 + \frac{M_{min}}{M_{max}} \times \frac{d_{max}}{d_{min}}$$

$$Z = 1 - \frac{N-1}{N} \times \frac{d''}{d}$$

$$f_s = \frac{3\dfrac{M}{Vd} - 0.45}{3\dfrac{M}{Vd} + 0.55}\left[6.9 \times 10^{-4} E_s \left(-1 + \sqrt{1 + \frac{1450}{NpE_s/k_1k_2f_c'}}\right)\right]$$

Shear moment capacities M_s' for beams without web reinforcement, computed with $k_2 = 0.42$ and $k_1k_3 = 1.121 - 4.85f_c' \times 10^{-5}$, are compared in Table 10 with the test data for beams failing in shear compression. The average ratio of the test to calculated values is 1.02, with a standard deviation of 0.066.

The shear moment capacities of beams with vertical stirrups in Shear Span 3 were computed from Eq. (7) given in Reference 7,

$$M_s = A_1M_s' + A_2A_3\, rf_{vy}\, b\,(a'')^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The constants A_1 and A_2A_3 to be used in Eq. (4) were determined from a plot of the ratio of ultimate test moment M_s (test) to calculated moment M_s' against $rf_{vy}\, b(a'')^2/M_s'$ as shown in Fig. 13. Fig. 13 also includes data obtained from the Moody and Viest[7] investigation.

TABLE 10—MOMENT AT SHEAR COMPRESSION FAILURE*

Beams without stirrups				Beams with stirrups							
Beam	Moment, in.-kips		M_s' test / M_s' calc.	Beam	Moment, in.-kips		M_s test / M_s calc.	Beam	Moment, in.-kips		M_s test / M_s calc.
	M_s' test	M_s' calc.			M_s test	M_s calc.			M_s test	M_s calc.	
E6N1	665	582	1.14	C6A1	928	768	1.21	E2A1	833	755	1.10
E6N2	509	497	1.02	E6H1	1005	936	1.07	E2A2	706	639	1.11
E6N3	580	564	1.03	E6H2	785	875	0.90	E2A3	753	668	1.13
C6N1	650	587	1.11	C6H1	875	979	0.89	C2A1	578	679	0.85
C6N2	478	472	1.01	C6H2	810	835	0.97	C2A2	705	642	1.10
C6N3	585	573	1.02	C6I2	730	897	0.81	E2H1	720	784	0.92
				E6I2	710	922	0.77	E2H2	590	658	0.90
								C2H1	815	771	1.05
	Average		1.06				0.95	C2H2	760	778	0.98
	Standard deviation		0.051	Average							
				Standard deviation			0.142				
E3N1	543	583	0.93					Average			1.02
E3N2	510	540	0.94	C3A2	931	843	1.10	Standard deviation			0.099
C3N2	519	528	0.98	E3H1	935	924	1.01				
				E3H2	860	949	0.91				
	Average		0.95	C3H1	815	815	1.00				
				C3H2	715	785	0.91				
	Over-all average		1.02					Over-all average			0.99
	Over-all standard deviation		0.066	Average			0.99	Over-all standard deviation			0.114
				Standard deviation			0.072				

*All moments pertain to the critical section at the edge of the center support bearing block.

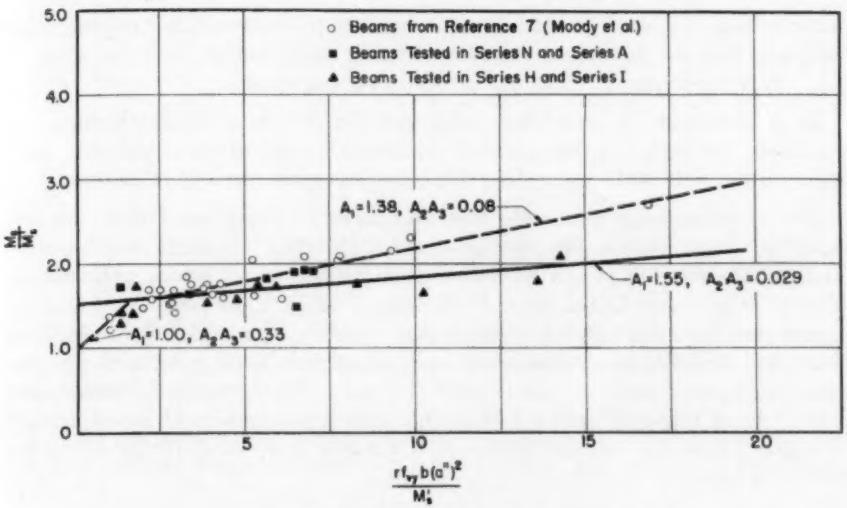


Fig. 13—Contribution of web reinforcement

The data from Moody and Viest[7] can be approximated by two straight lines with the constants

$$A_1 = 1.00 \text{ and } A_2A_3 = 0.33$$

or

$$A_1 = 1.38 \text{ and } A_2A_3 = 0.08$$

Data obtained from this investigation can be approximated by two straight lines with the constants

$$A_1 = 1.00 \text{ and } A_2A_3 = 0.33$$

or

$$A_1 = 1.55 \text{ and } A_2A_3 = 0.029$$

The difference in the two results cannot be thoroughly explained at present. However, Fig. 14 indicates that Eq. (4) is mathematically inadequate to express the shear capacity of reinforced concrete beams in general. Expressing Eq. (4) as

$$\frac{M_s}{M_s'} = A_1 + \frac{A_2A_3\, rf_{vy}\, b(a'')^2}{M_s'}$$

the ratio M_s/M_s' should approach a value of 1 as r approaches 0, regardless of the value of the distance a''. The data in Fig. 14 show that for beams reported by Moody and Viest[7] and beams of this investigation the curves do

not coverage to a single point but rather tend to follow a parallel pattern. This suggests that an important variable is being neglected or that the form of Eq. (4) is inadequate. This point merits further study.

It is important to note that, although Eq. (4) may be mathematically incorrect, the equation has yielded computed values of shear capacity that agree fairly well with test values for an appreciable number of beams.

The shear moment equations were compared in Reference 7 with the test data for simple beams obtained by Clark,[2] Moretto,[12] Richart and Jensen,[13] and by Moody *et al.*,[4] and with test data for restrained beams obtained by Moody *et al.*,[5] and Elstner *et al.*[6] Results of all of those tests were in good agreement with Eq. (3) and (4) of this investigation. The data obtained from this investigation show that essentially the same equations are applicable equally well to continuous beams. Furthermore, Morrow and Viest[10] have reported recently that the same equations yield safe ultimate strength values for simple beams with column stubs and for knee frames failing in shear.

Flexural capacity of beams

The flexural moment capacity of the positive moment sections has been designated as M^+ and the flexural capacity of the negative moment sections has been designated as M^-. From a consideration of the statical equilibrium of internal and external forces the limiting load carrying flexural capacity of beams tested in this investigation may be computed as

$$P_f = \frac{4}{L - 2e}\left[M^- + \frac{2(L - e)}{L - 2e} M^+\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

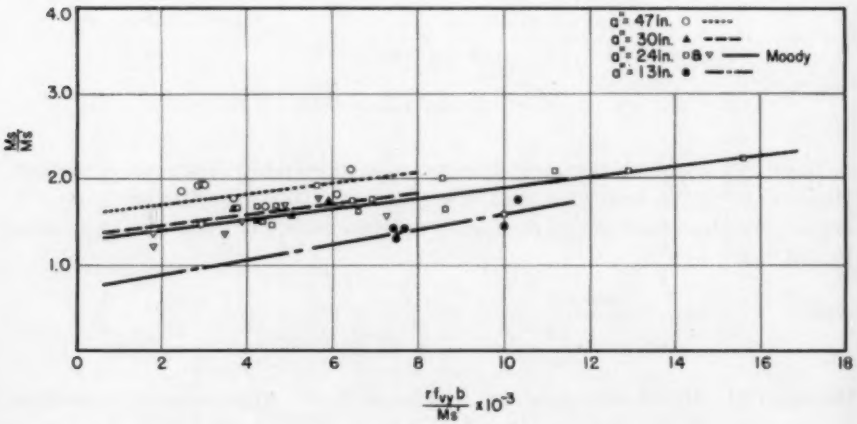


Fig. 14—Effect of a″ on the contribution of web reinforcement

for beams with ½-point loading,

$$P_f = \frac{4(L + 3e)}{L^2 - 2eL - 3e^2}\left[M^- + \frac{3(L - e)}{L + 3e}M^+\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

for beams with ⅓-point loading, and

$$P_f = \frac{4(L + 6e)}{L^2 - 2eL - 6e^2}\left[M^- + \frac{6(L - e)}{L + 6e}M^+\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

for beams with ⅙-point loading.* The quantity e is the distance from theoretical point of application of a load or reaction, to the edge of the load distributing block.

For the purpose of this paper, moment capacities M^+ and M^- were computed by the following expression obtained from Eq. (16) and (24) in Reference 14:

$$M_f = bd^2 f_c'\left[\bar{q}\left(1 - \frac{k_2}{k_1 k_2}\bar{q}\right) + \frac{A_s' f_s'}{bd f_c'} \times \frac{d''}{d}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where

$$\bar{q} = \frac{A_s f_y - A_s' f_s'}{bdf_c'}$$

$$k_2 = 0.42 \qquad \epsilon_u = 0.004$$

$$k_1 k_2 = 1.37 - 10.8 f' \times 10^{-5}$$

Resulting values for maximum positive and negative moments and for the limiting load capacity of beams failing in flexures are listed in Table 11. Agreement between computed and test values for the limiting load is very good. On the other hand, in most cases the test value for positive moment is larger than the computed value, while the test value for negative moment is smaller than the computed value. An explanation of the differences between computed and test values of positive moment may be found by an examination of strains in the bottom longitudinal reinforcement listed in Table 7. Note that in most beams failing in flexure the positive longitudinal reinforcement went into strain hardening before failure. Thus it appears reasonable that the

*Eq. (5), (6), and (7) are based on the assumption that moments M^- and M^+ are developed at the edges of the load distributing blocks. The negative moment M^- occurs at both edges of the block over the center support. The positive moment M^+ is developed in each span at the edge of block where the external moment is larger. For ½-point loading M^+ occurs at the outside edge of the loading block; for ⅓- and ⅕-point loadings it occurs at the inside edge of the outside loading blocks.

TABLE 11—FLEXURAL CAPACITIES

Beam	Maximum positive moment*			Maximum negative moment†			Limit load		
	M_f test, in.-kips	M_f calc., in.-kips	$\frac{M_f \text{ test}}{M_f \text{ calc.}}$	M_f test, in.-kips	M_f calc., in.-kips	$\frac{M_f \text{ test}}{M_f \text{ calc.}}$	P_f test, kips	P_f calc., kips	$\frac{P_f \text{ test}}{P_f \text{ calc.}}$
E6I1	735	670	1.10	750	961	0.78	210	209	1.00
C6I1	732	677	1.08	645	941	0.69	205	210	0.98
E3A1	645	623	1.04	925	998	0.93	120	118	1.02
E3A2	645	645	1.00	925	1030	0.90	120	122	0.98
E3I1	743	686	1.08	845	922	0.92	126	123	1.02
E3I2	729	659	1.11	845	942	0.90	124	120	1.03
C3I1	706	704	1.00	860	955	0.90	122	126	0.97
C3I2	703	639	1.10	895	978	0.92	123	119	1.03
E2I1	684	660	1.04	930	964	0.97	95	94	1.01
E2I2	714	653	1.09	930	991	0.94	97	95	1.04
C2I1	781	676	1.16	980	909	1.08	105	94	1.12
C2I2	711	659	1.08	875	938	0.93	95	93	1.02
	Average		1.07	Average		0.91	Average		1.02
	Standard deviation		0.045	Standard deviation		0.091	Standard deviation		0.037

*Positive moments pertain to critical section at the edge of outside loading block.
†Negative moments pertain to critical section at the edge of center support bearing block.

actual positive moment should be larger than the value computed on the basis of the yield point stress.

The low ratios of test to calculated values for negative moment could have resulted from two causes. In the first place, the diagonal tension cracks in Shear Span 3 extended close to the bottom beam surface near the center support and thus the compression zone may have been weakened. Secondly, the compression zone next to the center support was subjected, in addition to bending, to particularly high shear stresses; thus the principal compressive stresses above the bottom beam surface could have been more critical than compressive stresses along the beam surface. That the combination of the weakening effect of diagonal cracks and the high principal stress may have caused the difference between test and calculated values of negative moments is suggested also by the relative magnitude of the ratios M_f test/M_f calc.; these ratios are lowest for $\frac{1}{6}$-point loading and largest for $\frac{1}{2}$-point loading.

The smaller-than-actual computed values for positive moment and the larger-than-actual computed values for negative moment apparently compensated so that computed limit loads were in good agreement with test data. Note, however, that for combination of conditions different from those in this study the compensation may not be equally favorable. It appears, therefore, that the effect of shear on the moment capacity of a cross section needs further study.

Minimum amount of web reinforcement

The question of the minimum amount of web reinforcement needed to prevent shear failure was studied directly from test data and with the aid of Eq. (4) and (8). Data used for this purpose are summarized in Fig. 15 and in Table 12. Fig. 15 contains data for all specimens which failed in shear or in flexure, but the table contains only data for specimens with web rein-

forcement and with intermediate grade longitudinal steel which failed in shear or in flexure.

The test data shown in Fig. 15 suggests that for specimens tested the following percentages of web reinforcement were needed to prevent shear failures in beams with intermediate grade deformed bars: 0.65, 0.8, and 1.75 percent for the ½-, ⅓-, and ⅙-point loadings, respectively. Each of these percentages is higher than those required for the particular designs both by the 1951 and the 1956 ACI Codes. However, the percentages required by the 1956 Code for the ½- and ⅓-point loading are only slightly smaller than the minimum values determined from these tests.

TABLE 12—MINIMUM REQUIRED WEB REINFORCEMENT

Beam	Mode of failure*	Actual r, percent	Calculated r_{min}, percent	$\frac{r}{r_{min}}$
C6A1	S	1.33	2.79	0.48
C6I1	F	1.78	2.12	0.84
C6I2	S	2.22	2.41	0.92
E6I1	F	1.78	2.23	0.79
E6I2	S	2.22	2.39	0.93
C3A2	S	0.61	3.05	0.20
C3I1	F	0.95	1.41	0.67
C3I2	F	1.11	2.86	0.39
E3A1	F	0.61	2.12	0.29
E3A2	F	0.61	2.82	0.21
E3I1	F	0.95	1.06	0.90
E3I2	F	1.11	2.42	0.46
C2A1	S	0.37	1.72	0.22
C2A2	S	0.37	1.93	0.19
C2I2	F	0.67	2.10	0.32
C2I1	F	0.82	1.14	0.72
E2A1	S	0.37	1.51	0.25
E2A2	S	0.37	2.04	0.13
E2A3	S	0.37	2.07	0.18
E2I2	F	0.67	1.82	0.37
E2I1	F	0.82	1.66	0.49

*S—Shear failure
F—Flexural failure

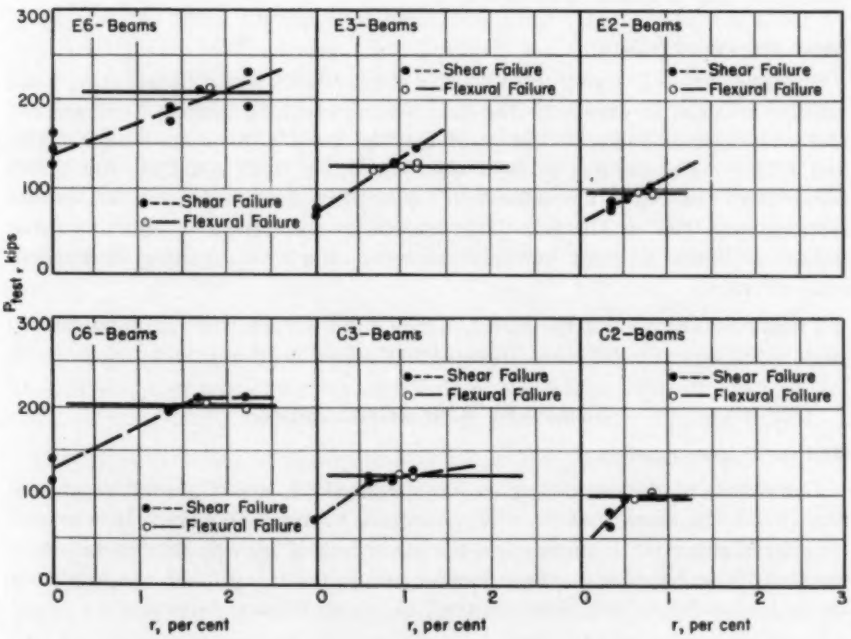


Fig. 15—Effect of percentage of stirrups on modes of failure

Note in Fig. 15 that for beams with cutoff negative reinforcement a certain limiting percentage of web reinforcement was reached for each type of loading, beyond which any increase in the percentage of web reinforcement appeared to be ineffective. No limiting value was noted in beams with extended negative steel.

The theoretical minimum amount of web reinforcement needed to prevent shear failure may be computed by equating the right hand sides of Eq. (4) and (8). Results of such computations are listed in Table 12 as r_{min}. It will be noted that specimens failing in shear had in all instances less web reinforcement r than this theoretical minimum amount. The ratio r/r_{min} for specimens failing in flexure varied between 0.21 and 0.84.

Eq. (4) consist of two terms: one considering the combined contribution of the compressive zone of concrete and the longitudinal reinforcement to shear strength of the beam, and the other estimating the contribution of web reinforcement alone. The comparison in Table 10 shows that Eq. (4) estimates reasonably well the over-all shear strength of the type of beam tested. It appears, therefore, that the equation overestimates the contribution of the beam itself and underestimates the contribution of the web reinforcement.

It must be concluded, therefore, that Eq. (4) is not suitable for determination of the minimum required amount of web reinforcement. Further studies of this problem are needed.

Bond stresses at failure

Specimens E2N1, E2N2, E2N3, C2N1, and C2N2 failed by splitting bond failures at loads in excess of diagonal tension cracking loads. Furthermore, test observations indicate the possibility of bond failures in Beams C3N1 and C3N2. The failures in Beams E2N1, E2N2, E2N3, C2N1, and C2N2 occurred in the top reinforcement in Shear Span 3 at nominal bond stresses ranging from 108 to 310 psi. This underlines the danger of splitting bond failures in beams without web reinforcement after the formation of diagonal tension cracks.

Failure of Beams E2H1, E2H2, C2H1, and C2H2 was accompanied by splitting of concrete adjacent to top longitudinal reinforcement.

SUMMARY AND CONCLUSIONS

Outline of investigation

The object of this investigation was primarily to study the effect of continuity on the shear behavior of reinforced concrete beams. It was also considered desirable to determine the contribution of web reinforcement to the shear strength of continuous beams, and to try to establish the minimum amount of web reinforcement required to insure flexural failures.

Tests were performed on 52 rectangular two-span continuous reinforced concrete beams, using three different arrangements of concentrated loads.

Positive moment reinforcement for all beams was provided in the amount required by ACI 318-51 for a balanced design; the negative moment reinforcement percentage was larger than that required by the ACI balanced design. In 25 beams the top reinforcement was cut off as permitted by ACI 318-51 while in 27 beams it was extended 20 bar diameters beyond the section of the maximum positive moment.

In the end shear spans of all beams more web reinforcement was provided than required by ACI 318-51. This web reinforcement proved sufficient to prevent shear failures in those spans. In all beams of Series A web reinforcement in Shear Spans 2 and 3 was provided in the amounts required by ACI 318-51; in beams of Series N no web reinforcement other than that in the end shear spans was provided. In beams of Series H the amount of web reinforcement in Shear Spans 2 and 3 was larger than that required by ACI 318-51, but still permitted shear compression failures. The beams of Series I were provided with enough web reinforcement to force the beams to fail in flexure before a shear failure could occur. All web reinforcement consisted of vertical stirrups.

Behavior of test beams

Four modes of failure were observed in these tests: diagonal tension, flexural, shear compression, and splitting failures.

Four beams failed by diagonal tension in locations at which ACI 318-51 required no web reinforcement, thus pointing out that the shear provisions for beams without web reinforcement were inadequate in these cases. In two beams without web reinforcement in the span adjacent to the center support, giving the lowest v_c/f_c' ratios encountered in these tests, diagonal tension cracks were observed to form at nominal shearing stresses of 193 psi and 152 psi for concrete strengths of 4390 psi and 3120 psi, respectively, corresponding to shearing stresses of 0.044 f_c' and 0.049 f_c'. The first of these two beams failed at a nominal shearing stress of 196 psi corresponding to 0.045 f_c'.

Twelve beams reached their full flexural capacity. Two of these had top longitudinal steel extended beyond the permissible cutoff points and had ACI 318-51 web reinforcement adjacent to the center support. All others had a larger amount of web reinforcement than required by ACI 318-51.

Thirty beams failed by shear compression in the shear span adjacent to the center support. Two of the beams with ACI web reinforcement failed in shear at the section adjacent to the load point. Both beams had the stirrups spaced at 10 in. instead of the maximum 6½ in. permitted by the ACI 318-51; in both cases the top reinforcement was cut off according to ACI 318-51. The corresponding beams with extended top steel failed at the section adjacent to the center support after yielding of web reinforcement. It appears then that close spacing of stirrups as required by the ACI Code is important,

especially if the top reinforcement is cut off according to the Code. The shear compression failure of four beams with web reinforcement was accompanied by splitting of the concrete along the top steel.

Conclusions

The following conclusions may be drawn from the results of the investigation of two-span continuous reinforced concrete beams.

1. Four modes of failure were observed in these tests: diagonal tension, flexural, shear compression, and splitting failures. Ultimate loads of beams that failed by the first three modes were in reasonable agreement with the strength analyses used for simple and restrained beams.

2. The analysis for diagonal tension cracking loads is based on the hypothesis that diagonal tension cracking results from excessive principal tensile stresses in regions already containing tension cracks. The equation for diagonal tension cracking loads used in analysis of these tests was developed from the study of simple beams with overhangs. Results of this investigation indicate that the analysis is applicable also to continuous beams.

3. Tests show that beams may fail simultaneously with the formation of a diagonal tension crack in a region in which no web reinforcement is provided. It appears then that the diagonal tension cracking load should be considered as the ultimate capacity for shear spans without web reinforcement. Table 6 indicates, however, that the shear span length influences the load capacity of the beam after formation of the diagonal tension crack. Formation of the initial diagonal tension crack has a less critical effect on load capacity of the shorter shear spans.

4. The calculated values for shear compression strength of continuous beams are in good agreement with test results. The analysis for shear compression strength is based on the hypothesis of a limiting shear moment capacity. It was developed from theoretical considerations in connection with tests of simple and restrained beams, and is known to yield safe values for stub beams and knee frames also.

5. The maximum flexural moment capacity of sections at which yielding of longitudinal steel was observed was in good agreement with the calculated values. The flexural moment capacity was analyzed with the aid of analytical expressions based on Bernoulli's hypothesis, and on properties of the stress block determined from tests of simple beams both with and without compressive reinforcement.

6. With the aid of equations for shear compression strength and for flexural moment capacity of reinforced concrete beams it was possible to predict satisfactorily the type of failure for beams with web reinforcement. Results for beams with cutoff negative steel seem to corroborate the findings reported in a previous investigation[15] that beyond a certain value an increase in the percentage of vertical stirrups does not increase the shear capacity of beams.

7. Values of the center reaction for all tests were in reasonable agreement with values computed from the elastic theory of structures. However, it is important to note that a small variation in the center reaction could affect the moment at critical sections appreciably.

8. This investigation showed that the manner of diagonal crack formation and the modes of failure of two-span continuous beams failing due to shear are similar to those observed in tests of simple beams and of simply supported restrained beams. However, continuity has a marked effect on redistribution of stresses in the beams. If yielding of longitudinal steel occurs at a section in a simple beam, failure has been almost reached. In a continuous beam longitudinal steel at a section may yield, but the beam can still carry additional loads.

9. Series A of this investigation indicates that the amount of web reinforcement required by ACI 318-51 was insufficient to prevent shear failure in the beams tested. Furthermore, several beams failed in diagonal tension in regions where no web reinforcement was required by ACI 318-51. Requirements for web reinforcement were changed in ACI 318-56. However, results of Series I indicate that these new requirements are still insufficient to prevent shear failures in beams of the type investigated.

10. Theoretically it appears possible to establish the minimum amount of web reinforcement required to insure a flexural failure by equating the flexural capacity formula to the shear capacity formula used in this investigation. However the results obtained by this procedure give values of r that are far too conservative. Therefore it is believed that additional studies are required to improve available mathematical expressions.

11. In beams tested in this investigation, cutting off negative longitudinal reinforcement as permitted by ACI 318-51 aggravated the critical situation brought on by an inadequate amount and spacing of web reinforcement.

12. Results of beams tested show that is is possible to obtain a shear failure in a beam even after yielding of the longitudinal reinforcement has occurred at a particular section.

ACKNOWLEDGMENTS

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The investigation was originally suggested by C. A. Willson, research engineer for the Committee on Reinforced Concrete Research, and by Eivind Hognestad, formerly research associate professor in the Department of Theoretical and Applied Mechanics.

JoDean Morrow, assistant professor; J. W. Baldwin, instructor; E. Bryant, senior laboratory mechanic; and M. R. Penny, junior laboratory mechanic—all of the department staff—participated in various phases of the test program.

The authors wish to express their gratitude to all who contributed to the success of this investigation.

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APPENDIX

After completion of the principal test program, four additional beams with increased concrete cover were tested. The purpose was to determine if a substantial increase in cover would prevent concrete splitting along the top longitudinal reinforcement which occurred in beams with ½-point loading tested in the main investigation. The fabrication of speci-

TABLE A-1—OUTLINE AND PROPERTIES OF BEAMS IN SUPPLEMENTARY TESTS*

Beam designation	Web reinforcement in Shear Span 3		Grade of longitudinal reinforcement	Yield point of reinforcement, psi			Concrete cylinder strength f_c', psi	
				Longitudinal		Web,		
	Size and spacing	Percent		Bottom	Top	Shear Span 3	Batch 1 and 2	Batch 3 and 4
B2N1	—	0	Intermediate	47,200	45,200		4000	3400
B2A1	# 3 @ 10 in.	0.27	Intermediate	49,000	50,700	51,200	3500	3100
B2H1	# 3 @ 4½ in.	0.59	Hard grade	64,700	62,500	47,600	3300	3300
B2H2	# 3 @ 7 in.	0.38	Hard grade	65,300	63,000	47,900	3800	3600

*All beams were 8¼ x 15⅛ in. in cross section and continuous over two 8.5-ft spans. Bottom reinforcement consisted of two #7 bars, the top reinforcement of two #9 bars, and the web reinforcement of #3 bar vertical stirrups. Effective depth of reinforcement was 13 in. for bottom steel and 12.5 in. for top steel.

mens, instrumentation, and test procedure for these tests were the same as explained in the main text of this report. Over-all length and size of longitudinal bars remained the same as in previous beams. The cross section was made 8¼ x 15⅛ in., with a top and side cover of 2 in., and a bottom cover of 1½ in. Clear distance between the top bars was 2 in. This change in cross section changed the percentage of longitudinal reinforcement from 2.67 to 1.96 percent for top steel, and from 1.60 to 1.09 percent for bottom steel. All beams had extended top longitudinal reinforcement. Size and spacing of stirrups was kept the same as for the beams in which splitting occurred. This means that the percentages of web reinforcement were decreased to values indicated in Table A-1. All beams were tested with one concentrated load applied in the middle of each span.

Beam B2N1, corresponding to Beam E2N1 in the main investigation, had no web reinforcement in Shear Span 3. Beam B2A1 was provided with the same size and spacing of stirrups as its corresponding beam, E2A1. Beams B2H1 and B2H2 had the same size and spacing of web reinforcement as Beams E2H1 and E2H2, respectively, and were provided also with hard grade longitudinal steel. All four beams behaved up to failure in a manner similar to that observed for the corresponding beams without additional cover.

All four beams failed by an extension of the main diagonal tension crack in Shear Span 3 along the top reinforcement, which caused clear splitting of the concrete. In most cases concrete in the compression zone next to the center support showed signs of crushing at failure.

For all four beams failure load could be estimated with good accuracy with the aid of Eq. (3) and (4) as shown in Table A-2. It should be noted, however, that the value of M^- test is always lower than the value of M^- calc. It seems possible that the presence of concrete splitting prevented development of the full shear moment capacity of the cross section. Nevertheless for these beams the reduction in load capacity was only slight.

It is concluded then that increased concrete cover did not change the basic mode of failure of these beams.

TABLE A-2—LOAD AND MOMENT AT FAILURE, SUPPLEMENTARY TESTS

Beam	Failure data				Center reaction, kips	Moment, in.-kips		$\frac{M_s \text{ test}}{M_s \text{ calc.}}$
	Diagonal tension cracking load, kips	Load P, kips	Location*	Mode†		M_s test	M_s calc.	
B2N1	45	64	3–T	S–B	43.0	518	550	0.94
B2A1	46	88	3–B	S	60.4‡	776	860	0.90
B2H1	50	125	3–T	S–B	82.8	950	988	0.96
B2H2	50	118	3–T	S–B	76.0	790	972	0.81

*3-T: Top of Shear Span 3 adjacent to load bearing block.
3-B: Bottom of Shear Span 3 adjacent to center support bearing block.
†S = shear compression, B = splitting bond failure.
‡Value computed by elastic theory. All other values were measured.

It is believed that the test data for the center reaction for Beam B2A1 are in error, which results in low values of M test for negative moment. If the center reaction is computed by the elastic theory of structures, the resulting value of M test for negative moment is in good agreement with the computed.

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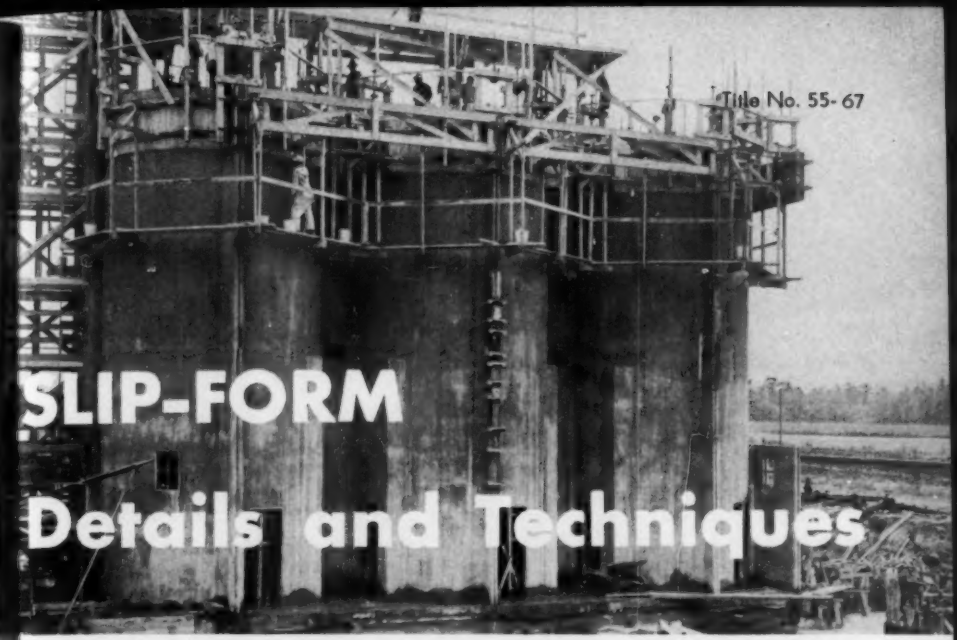

Title No. 55-67

SLIP-FORM Details and Techniques

A cursory description of the slip-form construction process—its advantages and limitations. Suggests practical considerations for good results: how to maintain a uniform supply of concrete and a uniform rate of placement; how to keep the construction deck level and walls in vertical alignment; techniques of placing the concrete; and problems of detailing and placing the steel. Surface finishes are also covered, and precautions for winter concreting indicated.

By J. F. CAMELLERIE

■ FOR MANY YEARS sliding-form construction has been closely associated with storage bins and silos. In this field "slip forms" have proved invaluable tools of construction, cutting cost and construction time, and at the same time enabling the work to proceed with maximum safety to the men. In recent years, however, aided by the perfection of fully automatic jacks and under the pressure of lively competition in the construction field, slip forms have been applied successfully to many other types of structures. We find bridge piers, apartment houses, water tanks, dam structures, and church steeples all being "slid." There is every indication that contractors and engineers will find successful application for sliding forms in the construction of many other types of structures.

In view of this, the author feels that a quick review of the nature, capabilities, limitations and techniques of this method may be of value now. To those engineers already acquainted with slip form, this paper offers nothing original, but perhaps it will stimulate some lively discussion. Information given here is mainly for those engineers who have only a nodding acquaintance with sliding-form construction and who are interested in seeing if it is

applicable to projects they now have under consideration and, if it is applicable, how they can best apply it.

No discussion is included here on static or dynamic pressures of fluids or granular materials. Many papers have already been written on this subject and work is in progress which will give us additional knowledge based on research. Nor will any other design considerations be discussed, since slip-form work does not in any way alter the design procedures and concepts in general use.

NATURE OF SLIP-FORM CONSTRUCTION

Slip-form construction can best be likened to an extrusion process. Plastic concrete is put into the top of the forms and the forms, acting as dies, shape this concrete so that it comes out of the bottom of the form hard enough to carry its own weight. The forms are 3½ to 4 ft high and are so constructed as to give the building its "plan" shape. Instead of concrete being pushed through the die, the forms are pushed upward to receive new concrete at the

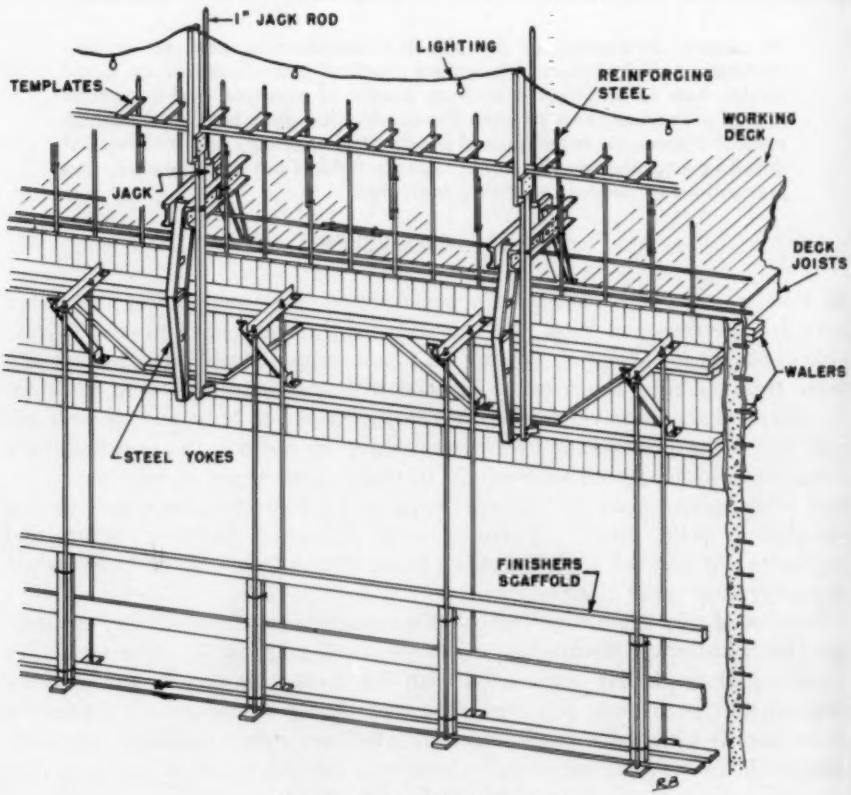

Fig. 1—Typical slip form deck

top. Locomotion is accomplished by jacks riding on 1-in. smooth steel rods embedded in the hardened concrete below (Fig. 1). These jacks may be manual, pneumatic, electric, or hydraulic and are capable of speeds up to 20 in. per hr. Normally, jacks of 3-ton capacity are adequate but much higher capacities are available as required. Working decks and finishers' scaffolds are attached to and carried up with the forms (see Fig. 1). The raising and filling of the forms is usually a continuous process carried on 24 hr a day until the structure has reached the top. Using sliding forms, walls and piers may be cast at a rate of 12 to 24 ft per day, depending on job requirements and conditions. Although continuous slides are desirable, the sliding operation may be stopped and later resumed with the resulting joints no different from those between lifts of fixed-form construction.

As vice-president and chief engineer of Plummer Associates, New York, J. F. CAMELLERIE meets many engineers, architects, and contractors vitally interested in slip-form construction, but lacking practical, "down-to-earth" and "up-to-date" facts on its general applications. With this in mind, he has drawn from his own experiences and those of his colleagues to prepare this paper. An ACI member who serves on Committee 714, Concrete Bins and Silos, Mr. Camellerie has been active for more than a decade in the design and construction of structures using the slip-form method.

Projections beyond face of forms

Since slip-form construction is an extrusion type of process, no projections beyond the face of the forms may be allowed as any such projection will foul the moving forms. If such projections are required, they may be added after the slide in one of several ways. For example, for a concrete bracket, pockets may be left in the wall with dowels bent in so as not to project beyond the face of the wall (Fig. 2). After the slide is completed, the dowels are bent out and lapped or welded to the bracket steel; the bracket is then cast in place. If a steel bracket is to be used, anchored weld plates are cast into the wall during the slide with the face of the plate flush with the face of the wall. The steel brackets may later be welded to these plates. Horizontal anchor bolts may not be allowed to project beyond the face of the walls. Weld plates, inserts, slots, or cinch anchors may be substituted. Unless inserts are rigidly attached to a nonmoving object, a steel wall collar or forming for a wall opening, their use is not recommended because of the difficulty of holding them in proper position as the forms slide by.

Wall openings, architectural features

Although projections beyond the wall faces are not feasible during the actual slide, a great deal of versatility is possible within the confines of the forms (Fig. 2). Entire sections of wall may be blocked out and resumed as required using stationary or moving bulkheads that fit between the forms. By the use of these bulkheads, walls may be stepped down, pilasters may be

picked up or dropped, and beams and columns may be cast without difficulty. Windows and door frames may be cast right into the walls as the slide proceeds, and normally no difficulties are encountered in placing piping or electrical conduits in the walls or columns. Pockets to take beams, keys with dowels, and anchor slots for masonry attachment are also placed during the slide.

Savings of as much as 25 percent have been attributed to the use of sliding forms. Naturally, the placing of welding plates, inserts, openings and pockets, changes in wall locations and sizes, and other complications of this sort do reduce the speed of the slide which, in turn, proportionately reduces the savings inherent in this method. Lack of height is another factor which may reduce the economy of slip-form construction. Structures less than 40 ft high do not usually lend themselves to the economical use of sliding forms.

Walls can be stepped but not tapered. Horizontal grooves should be avoided as the strips required to make these grooves are difficult to hold in place and keep level. Vertical grooves, fluting, and any other architectural feature of a vertical nature offer no difficulty. Horizontal architectural features within the first 4 ft and, of course, at the top of the slide are entirely feasible. There is no reason why slip-formed structures cannot be constructed with the above considerations in mind while preserving the aesthetic beauty of the structure.

Horizontal structural members

Floors, roofs, hoppers, landings, etc., are placed after the slide is completed, using the usual fixed-form methods. There are, however, certain

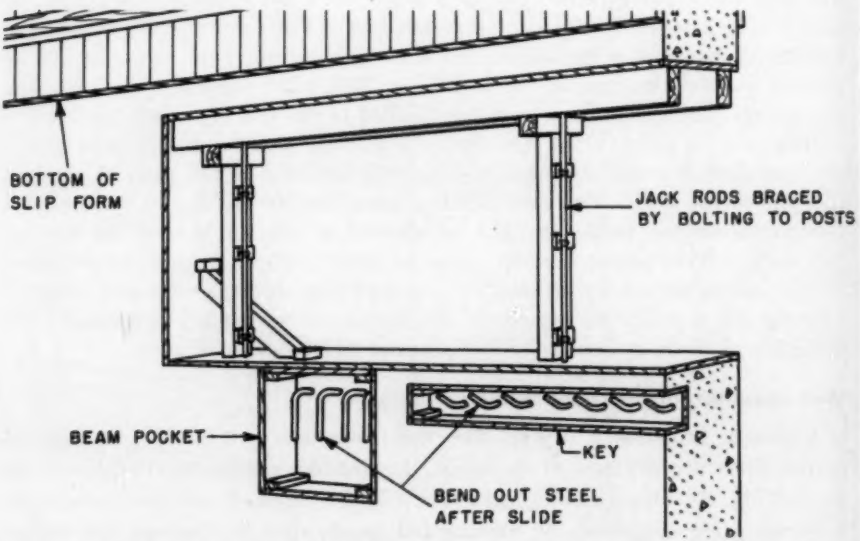

Fig. 2—Typical forming details allow for openings, beam pockets, concrete brackets, etc.

advantages and possibilities which should be considered. The working deck can usually be used to place the roof, thus saving the formwork and shoring usually required and facilitating work on the floors below by closing in the building to the weather. Often beams and columns can be cast with the slide, and a concrete slab is placed later on these beams with the formwork supported by the beams. This is also a good spot for precast slabs.

ESSENTIAL CONSTRUCTION PRACTICES

Planning, equipment, and supervision

Since the economic success of the slide operation depends primarily on attaining the highest possible rate of slide consistent with good construction practice, careful planning and supervision are necessary to minimize delays. Some items which materially affect the rate of slide and which should be carefully planned before the work is begun are the following:

1. An adequate supply of concrete to maintain an average rate of slide of 12 in. per hr. This requires an all-weather road and a reliable source of concrete or an adequate batch plant with sufficient materials stockpiled. Hoist tower, crane, Archer hoist, Chicago boom, or other means of vertical transport of concrete must also be of sufficient capacity.
2. A crew of concrete laborers, finishers, iron workers, carpenters, and electricians sufficient to maintain the rate of slide. Care should be taken that crew is not so large as to congest the working deck and thus decrease efficiency.
3. Competent supervisors having adequate experience with sliding forms are a must.
4. A good concrete mix and careful control to maintain proper slump in the face of changing temperatures and conditions.
5. Adequate hoisting facilities for reinforcing steel, formwork, iron work, etc.
6. Standby hoist, pumps, buckets, trucks, etc., for use in the event of a breakdown.
7. Formwork and iron work prepared for placing in the forms before the slide is started.
8. Forms designed to sustain stresses resulting from the liquid head of concrete to be expected at the maximum rate of slide and at the temperatures that will be encountered.

Although the maximum rate of slide must be carefully predicted and planned for, the actual rate must be controlled by an experienced field superintendent who is qualified to make such adjustments as changing field conditions inevitably require. The superintendent must see that the slide operation is not proceeding so fast as to cause "blowouts" or soft concrete falling out from under the forms, nor so slow as to cause "lifts" or concrete sticking to forms and ripping away from the concrete below.

Maintaining horizontal and vertical alignment of forms and deck

Another important consideration peculiar to this type of construction is maintaining the deck level during the slide. The following methods are recommended to check level.

1. Check position of jacks against hacksaw cuts previously made in jack rods at 1-ft intervals.

2. Use of a water-level system which consists of a central reservoir and plastic tubes placed at strategic locations and connected to the reservoir by hose.

3. Check by transit on deck. This can only be done while deck is stationary and because of this the use of transit is limited. Such a check can and should be made at the end of the slide for final leveling.

4. Vertical tape measures from fixed points. This method is again only usable while the forms are stationary but, less time is required if not too many points are checked.

Corrections may be made by operation of individual jacks or by keeping certain jacks stationary while the others are rising. This same method can be used to a limited extent in correcting discrepancies in plumbness. Correction for plumbness is difficult and can be made only under the supervision of a man highly experienced in slip-form work. The forms must be set up plumb and true and properly braced. The forms must be evenly filled when the slide is first started and during the slide. The deck must be kept level throughout the slide. If these three things are done, there will be no chance of the structure going out of plumb. A tolerance of 1 in. per 50 ft of height should be allowed.

The walls or piers should be periodically checked for plumbness as the work progresses. The following are some of the methods used:

1. Targets painted on at least three sides of the structure in sets of two. Each set consists of one target painted on the moving form and one target vertically below painted on the base of the wall. Periodic sights are taken to insure the line connecting the two targets remains vertical.

2. Plumb bobs dropped from several predetermined points on the deck to points previously marked on the base slab.

3. Check measurements between center lines, diagonals, etc.

PLACING CONCRETE

The sliding-form method requires no special types or mixes of concrete and placing of the concrete is governed by the same principles that govern the placing of concrete in fixed forms. However, there are some considerations involved which are peculiar to sliding-form work.

Concrete is placed in horizontal layers of from 6 to 10 in. in thickness, the 6-in. layers being the most desirable. The slump should be between 3 and 6 in. depending on temperature, wall thickness, reinforcing congestion, and anticipated rate of slide. Care must be taken to spade concrete thoroughly. Mechanical vibrators of the immersion type may be used but generally hand spading is preferable as great care must be taken that the vibrator is not forced into concrete that has already had some set. Note too that forms must be specially designed to take the additional pressure that results from mechanical vibration, and that the relatively higher slump increases the danger of separation. Control of concrete slump should be placed in the hands of the field engineer or superintendent who should, in turn, vary the slump without change to the water-cement ratio. Aggregate

size should be limited to ¾ in. for walls less than 8 in. thick and 1 in. for walls 8-12 in. thick. Walls less than 6 in. thick are not recommended because of difficulty in spading.

Not more than 1 hr should elapse between the placing of successive layers of concrete; this will insure proper bonding between layers and prevent spading of partially set concrete. If delays occur, concrete should be placed in layers less than 6 in. thick rather than allow the time between placing successive layers to exceed 1 hr. Set may be retarded by substituting crushed ice for water or accelerated by heating the water.

Decks must be kept broom clean at all times and concrete must not be allowed to accumulate around forms. Any concrete or debris that does accumulate on the deck should be swept away from the forms and removed from the deck.

DETAILING AND PLACING OF REINFORCING STEEL

The detailing and placing of reinforcing steel varies from standard practice only in the following respects. Vertical bars are limited in length to avoid excessive lash from the wind. The following limitations in length of vertical bars are suggested:

½-in. bars	14 ft
⅝-in. bars	17 ft
¾-in. bars	20 ft
⅞-in. bars	22 ft
1-in. bars	25 ft

Verticals over 20 ft long should not be used unless a second deck is planned over the main deck for use by the reinforcing crew, since bars weighing more than 35 lb are hard to handle from the bottom end.

Horizontal bars should also be limited in length to enable the steel to be threaded through the vertical steel and the form yokes. Lengths should generally be limited to 18 or 20 ft, and when hooks are required 10 ft is a better figure. The use of 90-deg hooks instead of standard hooks considerably facilitates placing. In circular bins, the laps in the hoop steel should be staggered, while in straight walls subject to moments, splices are made at the points of inflection. It is good practice to make horizontal bars 6 in. longer than is required by ACI 318-56 to insure against accidental inaccuracy in placing.

During construction vertical steel must be held in place by appropriate templates and care should be taken that required cover is maintained. Whenever tying of the vertical steel in baskets is possible, as in the case of columns, the first run of such steel should be so tied. Such baskets resting on the base slab may be as much as 35 ft high. Positive measures must be taken to keep these baskets positively plumb during the slide. Note that this basket arrangement requires that the jack yokes be so placed as not to foul the tie steel. Verticals must be omitted where runways for concrete buggies

are required. The runway must be at least 3 ft 6 in. wide, but 4 ft is preferable. If the engineer feels it necessary, additional vertical bars may be grouped around the runway to make up for bars omitted.

Some methods used to insure proper placing of horizontal steel are as follows:

1. Saw cuts in vertical steel indicating the horizontal spacing.
2. After a layer is placed a few of the bars of the next layer may be tied in place (where the yokes allow) as guides for the next layer.
3. Concrete may be placed in layers equal to the spacing of the horizontal steel. The steel is then placed as soon as each layer of concrete is in place. The next layer of concrete may not be placed until the reinforcing is in place.
4. To facilitate placing and inspection, horizontal steel should be designed in full layers and different spacing of horizontal steel at any one level should be avoided.
5. Use of double-headed nails or other markers nailed into the splash board or deck to indicate end of rods, thus saving measuring for each course. This, of course, cannot be done in deck areas subject to traffic.

In detailing vertical reinforcing steel, especially the larger size bars, splices should be staggered at various levels so as to equalize the burden on the iron worker crews. Whenever possible, horizontal steel should be spaced at 6-18 in. on centers with the spacings nearer 12 in. being the most desirable. Bars ¾ in. in diameter and smaller are generally preferable to larger sizes.

CURING AND FINISHING "SLID" SURFACES

Finishing provides no problems in sliding-form construction. The concrete coming out from under the sliding form is in excellent condition for a float and brush finish. The finishers work comfortably from the hanging scaffold (Fig. 1). This, together with the absence of joints, results in a finish that is durable when exposed to the weather.

One curing method is the use of membrane-type curing compounds applied directly from the finishers' scaffold. Another method is water curing using water lines hung from the forms. If water curing is used, fog spray is recommended to prevent erosion or discoloration.

WINTER CONCRETING

Slip-form operations can be, and often are, conducted at temperatures below 40 F. To insure a high quality concrete, the following special provisions are suggested:

1. Use of Type III air-entraining cement.
2. Heating of concrete to 70 F in the forms.
3. Use of protective enclosure capable of maintaining air around green concrete at 70 F for 24 hr.
4. Use of reinforcing steel that is free of ice.

These provisions are only one possibility among many. It is possible to use Type I or Type II cement or to keep the air at only 50 F, but this would require a minimum curing period of 3 to 5 days. If the slide is limited to

6 in. per hr, a 3-day curing period is equivalent to 36 ft of slide. It is doubtful that a 36-ft enclosure can be hung from the sliding forms without serious jeopardy to the forms and certainly there will be a tendency to pull the forms out of plumb. The other alternative is to build an enclosure on the ground and to build it up higher as the slide progresses. For high structures this method is quite expensive.

On the other hand, if the protection time is kept down to 24 hr, the height of the enclosure need be only 12 ft. This much can be hung from the sliding forms with safety and can follow the forms to the top of the slide without requiring carpenters to work in the open at great heights to erect the enclosure. This safety feature and the economy of construction and convenience in operation, justify the use of 24-hr curing and the provisions suggested above.

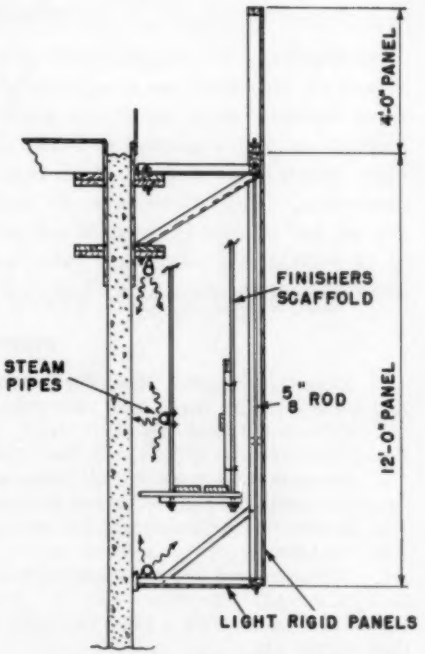

Fig. 3—Rigid enclosure for winter slip form work

The most common enclosure used on slip-form work consists of tarpaulins hung from the deck hand rails down over the finishers' scaffolds. This method has been satisfactory on many jobs. Nevertheless, the author favors an enclosure of rigid panels (plywood, masonite, insulation board, etc.) hung from the forms by separate brackets as shown in Fig. 3. By using separate brackets, the enclosure is free of the finishers' scaffolds thereby not shaking the scaffolds in gusty winds. If the wind should damage the enclosure and cause a section to fall off, this section will not drag down the scaffold (and possibly men) with it. The panels are superior to the tarpaulins in that they do not flap in the wind, they hold the warm air better, and will not tend to come apart during the slide.

It is suggested that steam heat be used so as not to dry out the concrete, both on inside and outside surfaces. As the inside areas of slip-form jobs tend to be warm and humid by virtue of their protected location, a steam source at the base of the slide is all that is usually needed. The warm, moist air rises through the building maintaining excellent curing conditions. The outside walls are cured by spraying on a membrane-type curing compound after the walls are finished and before they emerge from the enclosure. Every effort should be made to equalize the temperatures on the inside and outside faces of green walls to prevent temperature cracks.

CONCLUSIONS

Sliding-form construction will under certain conditions result in durable, attractive structures at substantial savings in cost and in time. To realize these benefits, such structures must be designed with the capabilities and limitations of this method in mind. Generally speaking, buildings over 40 ft high, which have straight, clean lines and which are free of projecting ornamentation, lend themselves to this method. Sliding-form construction can be and should be used in conjunction with other construction methods so as to reap the advantage inherent in each method and thereby increase construction efficiency as a whole.

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Title No. 55-68

High-Density Concrete

Made with Hydrous-Iron Aggregates

By Harold S. Davis and Orville E. Borge

DATA ARE PRESENTED on the physical properties of a high-density concrete tested under standard conditions and after heating at temperatures of 85, 200, and 350 C. The concrete has a density of 218 lb per cu ft and a fixed water content of about 4.3 percent at 85 C, which makes it a desirable material for biological shields around atomic power reactors. Computed values of attenuation lengths are included for fast neutrons and gamma radiation.

CONCRETE IS AN ECONOMICAL AND EFFECTIVE MATERIAL for shielding atomic power plants and other types of stationary reactors. Shield thicknesses can be reduced by utilizing high-density concrete instead of ordinary (150 lb per cu ft) concrete. Concrete weighing around 230 lb per cu ft is attainable if made with magnetite, ilmenite, or certain other aggregates. Limonite is sometimes blended with magnetite or steel punchings in order to increase the amount of water (hydrogen) available for attenuating neutrons.[1] The hydrous-iron ore utilized in this test program is of special interest since it combines high density, a moderate water content, and excellent strength in one aggregate. Its use eliminates the need for blending limonite and magnetite aggregates during construction, and indicates a potential cost reduction, depending on the geographical location of the nuclear reactor.

The tests described in this report were conducted to determine the physical properties of high-density concrete made with a hydrous-iron aggregate obtained from central Montana. In addition to the normal tests made under standard conditions, tests were also performed on concrete specimens which had been heated at a temperature of 85, 200, or 350 C for 2 weeks.

This concrete, designated as Concrete G, is the seventh type of concrete investigated at the North Pacific Division Laboratory by the Corps of Engineers, United States Army, for the Atomic Energy Commission. Tests performed previously on Concretes A to F are described in Reference 2.

MATERIALS

Mix data

This high-density concrete was made with 7.0 sacks per cu yd of Type II portland cement with 1.0 percent densifier (by weight of cement). Water-cement ratio was 0.43 by weight. Heavy aggregates processed from hydrous-iron ore were 46.5 percent fine and 53.5 percent coarse. Slump was 2¾ in., air content 1.9 percent, and unit weight, wet, 216.4 lb per cu ft.

The workability of the concrete was good but it lacked plasticity. Little effort was required to consolidate it in the molds. However, bleeding and segregation were greater than would be expected for a comparable mix made with conventional aggregates. The fresh concrete consolidated readily when moved in a pneumatic tired wheelbarrow, indicating a possible problem when transporting it.

Cement

The Type II portland cement was sampled and tested by a National Bureau of Standards Laboratory (Laboratory No. SFA5074; July 14, 1955). The results of the cement tests are summarized in Table 1.

TABLE 1—PROPERTIES OF PORTLAND CEMENT

Item	Percent	Item	Percent	Item	Amount
Ignition loss	1.4	C_3S	46.0	Specific surface, sq cm per g, Blaine method	3370
Insoluble residue	0.0	C_2S	30.0		
SO_3	2.2	C_3A	4.0		
MgO	2.0	C_4AF	12.0	Compressive strength, psi	
SiO_2	22.7	Alkalies as Na_2O	0.44	3 days	2520
Al_2O_3	3.9	Entrained air	2.9	7 days	3290
Fe_2O_3	3.8	Autoclave expansion	0.02	28 days	5250

TABLE 2—QUANTITATIVE AND QUALITATIVE ANALYSES OF HYDROUS-IRON ORE SAMPLE

Property	Quantitative analysis				Qualitative analysis†			
	Sample No.				Element	Approximate concentration	Element	Approximate concentration
	1	4	6	7				
Specific gravity	4.4	4.5	4.2	4.4	Ag	T	Mg	M
Percent iron	63.7	59.8	52.9	63.8	Al	M	Mn	T-M
Percent silica	—	5.2	—	—	As	T	Na	T
Ignition loss* at:					Ba	T-M	Ni	T-M
200 C	0.1	0.3	—	—	Be	T	Pb	M
500 C	4.3	4.5	2.7	4.8	Bi	T	Si	M to M-S
Description	—	—	Lustery grey, some crystals	Dull grey brownish, earthy	Ca	M	Ti	T-M
					Cu	T-M	V	T
					Fe	S	Zn	M
					K	T-M		

NOTE: Tests made at Hanford Atomic Products Operation on hand-picked Running Wolf ore samples obtained prior to procuring iron ore used in concrete test program.

*Considered to be mostly water of hydration.

†Spectrochemical analysis: S (strong) greater than 1 percent concentration; M (moderate) 1 to 0.01 percent; and T (trace) less than 0.01 percent.

Running Wolf hydrous-iron ore

The heavy aggregates were processed from mine-run ore obtained from the Running Wolf iron deposits located about 20 miles southwest of Stanford, Mont. In general, the ore is a fine-grained mixture of magnetite, hematite, and limonite, with magnetite predominating. (Running Wolf ore samples, reported by the Bureau of Mines, contain 58.75 percent iron, 1.109 percent sulfur, 0.02 percent phosphorus, and 9.53 percent insoluble.[3]) Typical data for hand-picked samples tested at Hanford Laboratories are summarized in Table 2.

Mine-run ore from two adjacent claims was delivered to the laboratory and crushed separately. A well graded, 1.5-in. maximum size aggregate was produced in jaw and gyratory crushers set for 1.5-in. and ¾-in. maximum openings, respectively. Sand was produced from the minus No. 4 crusher fines, or from surplus ¾-in. to No. 4 material, by means of a rodmill and a rake classifier. Gradations and physical data for the processed aggregates are presented in Tables 3 and 4. Individual pieces of coarse aggregate are fairly cubical in shape, but with sharp, irregular edges, and hard to break.

METHODS

In general, standard methods were used for preparing specimens and performing physical tests. Procedures used by the Corps of Engineers are described in Reference 4. The various CRD numbers and corresponding ASTM designations are summarized in Table 5 for the tests performed during this program.

The wet concrete was consolidated in the molds by means of a vibrator in accordance with Test Method CRD-C-10. The specimens were allowed to cure in their molds at 23 C and a relative humidity of 95 to 100 percent for 48 hr, after which they were stripped and placed in the fog room again until 28 days old. At 28 days, the specimens were removed from the fog room and stored in the laboratory (relative humidity about 50 percent) until tested and heated. Tests on the hardened concrete were performed

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TABLE 3—GRADATION OF PROCESSED AGGREGATES*

Sieve size	Percent passing		
	Sand	¾ in.—No. 4	1½ in.—¾ in.
2 in.			100.0
1½ in.		—	99.0
1 in.		100.0	46.0
¾ in.		96.9	9.6
½ in.	—	54.0	0.5
⅜ in.	100	32.4	—
No. 4	96.1	2.6	
No. 8	80.1	—	
No. 16	59.0		
No. 30	38.2		
No. 50	22.1		
No. 100	9.7		
Fineness modulus	2.95		

*Crushed ore from the first and second shipments was blended to obtain processed aggregate used in concrete (see text).

after 28 days of moist curing, after an additional 62 days of storage in the laboratory air (90 days old), and after subsequent 2-week periods of storage at 85, 200, or 350 C to determine the effect of static heating. Three specimens of each type were tested at 28, 90, and about 106 days, respectively, except only one water permeability and two diffusivity specimens were tested. Heated specimens, 106 days old, were allowed to cool to room temperature just prior to being tested in compression, bond, or flexure.

Water permeability was obtained by means of a radial flow, falling-head permeability test, the head being 12.9 to 10.9 ft. The specimen consisted of a hollow cylinder, 18 in. high, having a 9-in. outside diameter and a 4⅜-in. core. The test was run with a concrete specimen which had been cured 90 days in the fog room.

RESULTS

The results of the physical tests performed at standard and elevated temperatures are summarized in Table 6.

The water permeability was found to be 1.24×10^{-10} ft per min for the 90-day specimen. The specific heat was determined to be 0.20 Btu per lb per deg F, based on values of specific heat for the coarse aggregate and a representative amount of hardened mortar. The thermal conductivity

TABLE 4—PHYSICAL PROPERTIES OF HEAVY AGGREGATES

Property	Sand	First shipment		Second shipment	
		¾ in. to No. 4	1.5 in. to ¾ in.	¾ in. to No. 4	1.5 in. to ¾ in.
Specific gravity, bulk, saturated surface dry		4.30	4.40	3.99	4.10
Absorption, percent		1.3	0.9	0.9	0.5
Los Angeles abrasion, percent loss at 500 revolutions (Grading A)		25.3		17.7	
Unit weight (plus No. 4), lb per cu ft, dry rodded		150.5		142.9	
Thermal expansion, in. per in. per deg C × 10⁶	10.7 (Mortar)		9.2		8.8
Weight loss in heating, percent, at:					
85 C	0.1	0.1		0.1	
200 C	0.4	0.2		0.2	
350 C	2.0	3.6		2.9	
600 C*	1.9	3.8		3.4	
900 C*	1.2	3.8		3.0	

*Some oxidation occurs at high temperatures which may cause an increase in weight.

TABLE 5—SUMMARY OF TEST METHODS

Test	CRD* number	ASTM† designation	Remarks
Aggregates:			
Gradation	C-103-48	C-136-46	Blended aggregate
Unit weight	C-106-51	C-29-42	Dry rodded
Abrasion	C-117-56	C-131-55	Los Angeles, Grading A
Specific gravity and absorption	C-107-51 C-108-48	C-127-42 C-128-42	Coarse aggregate Fine aggregate
Thermal expansion	C-125-52	—	Strain gage method
Plastic concrete:			
Test specimens	C-10-52	C-192-52	Cured at 73 F and relative humidity of 95 to 100 percent
Air content	C-41-52	—	0.25 cu ft "Pres-Ur-Meter"
Consistency	C-5-52	C-143-52	Slump cone
Hardened concrete:			
Compression	C-14-54	C-39-49	6 x 12-in. cylinders
Modulus of elasticity	C-19-55	—	6 x 12-in. cylinders
Bond	C-24-48	C-234-49T	9 x 9 x 9-in. cube and ¾-in. bar
Flexure	C-16-54	C-78-49	6 x 6 x 24-in. beams
Compression	C-15-54	C-116-49	6 x 6 x 12-in. (modified cube)
Length and weight change	C-25-48	C-157-43	6 x 6 x 24-in. beams
Thermal expansion	C-39-55	—	6 x 6 x 24-in. beams
Thermal expansion	C-126-52	—	2 x 2 x 11-in. mortar bars
Diffusivity	C-36-48	—	6 x 12-in. cylinder
Specific heat	C-124-51	—	Coarse aggregate and mortar

*Corps of Engineers, U. S. Army, *Handbook for Concrete and Cement*, Waterways Experiment Station, Vicksburg, Miss.

†*ASTM Book of Standards, Part 3*, American Society for Testing Materials, Philadelphia, Pa.

TABLE 6—SUMMARY OF TESTS ON HIGH-DENSITY CONCRETE

Properties	Specimens at room temperature		Heated specimens		
Age, days Storage condition*	28 Moist	90 Laboratory air	106 85 C	106 200 C	106 350 C
Unit weight, lb per cu ft					
Beams	224.13	221.40	218.41	215.52	208.86
Cylinders	223.04	221.09	—	216.73	210.34
Compressive strength, psi 6 x 12 in. cylinders	6910	8010	—	8040	6850
Modulus of rupture, psi 6 x 6 x 24-in. beams	965	930	1015	930	840
Modified cube compression, psi (ends of 6 x 6 x 24-in. beams)	8440	10,700	7340	6900	6920
Bond strength†, psi, ¾-in. bar, 9-in. cube at slippage					
0.00005 in.	1200	1120		930	320
0.0001 in.	1320	1180		1010	400
0.0002 in.	1420	1370		1220	570
0.0003 in.	1520	1420		1290	690
0.0005 in.	1550	1430		1390	880
0.0010 in.	1560	(Steel		(Steel	1300
0.0015 in.	1580	failed)		failed)	(Steel failed)
Thermal diffusivity, sq ft per hr, 6 x 12-in. cylinder		0.032	0.034	—	0.033
Dimensional stability 6 x 6 x 24-in. beams,					
Length change, percent	0.002	−0.006	−0.020	−0.058	−0.111
Weight change, percent	0.4	−1.1	−2.4	−3.8	−6.2
Coefficient of thermal expansion in. per in. per deg C × 10^6, 6 x 6 x 24-in. beams	9.35	9.93	9.50	9.07	8.25
Modulus of elasticity, psi × 10^{-6}, 6 x 12-in. cylinders at:					
1000 psi stress	7.52	8.29	—	5.66	3.58
2000 psi stress	7.40	8.47	—	5.78	3.81
3000 psi stress	7.42	8.33	—	5.83	4.00

*See text. Standard temperature beams stored moist 2 days weighed 223.53 lb per cu ft. Cylinders under same condition weighed 222.46 lb per cu ft.

†Ultimate bond strength of concrete exceeded strength of reinforcing steel rod.

for the 90-day specimens can be computed as follows from the diffusivity, specific heat, and weight per cu ft:

$$k = 0.032 \times 0.20 \times 221 = 1.42 \text{ Btu per ft per hr per deg F}$$

Diffusivity values for the 85 C and 350 C tests were determined with heated specimens which had been cooled and soaked prior to running the diffusivity tests. Thus, values of thermal conductivity and diffusivity for dry, heated specimens are probably less than indicated above.

DISCUSSION

As shown by the test data summarized in Table 6, the concrete possesses excellent structural properties, exceeding those of concretes made with iron-bearing aggregates reported in Reference 2. This could be expected since a 7-bag mix was used in combination with a very strong aggregate. Heating at temperatures of 85 and 200 C for 2 weeks produced a small amount of deterioration in structural properties. However, deterioration following the 350 C storage period was somewhat greater than for either of the other heating tests. In this case, bond strength at small slippage values decreased about 70 percent and the modulus of elasticity about 50 percent (see Table 7).

Using the data presented above, it is possible to estimate the amount of water retained after the concrete was heated to constant weight at a particular temperature. Computed values of water content are summarized in Table 8, along with attenuation data for fast neutrons and hard gamma radiation. Unit weight data compare favorably with measured values considering the uncertainties involved in ascertaining water content and chemical composition. It is noted that the attenuation effectiveness for gamma radiation decreases almost directly with the unit weight of the heated concrete, whereas

TABLE 7—EFFECT OF HEATING ON PHYSICAL PROPERTIES*

Properties	Specimens at room temperature		Heated specimens		
Age, days Storage condition (see Text)	28 Moist	90 Laboratory air	106 85 C	106 200 C	106 350 C
Unit weight					
Beams	101	100	99	97	94
Cylinders	101	100	—	98	95
Compressive strength, 6 x 12-in. cylinders	78	100	—	90	77
Modulus of rupture, 6 x 6 x 24-in. beams	104	100	109	100	90
Modified cube compression, ends of 6 x 6 x 24-in. beams	79	100	69	64	65
Bond strength, ¾-in. bar, 9-in. cube, at slippage of:					
0.0001 in.	112	100	—	86	34
0.0005 in.	108	100	—	97	62
Coefficient of thermal expansion, 6 x 6 x 24-in. beams	94	100	96	91	83
Modulus of elasticity, 6 x 12-in. cylinders at:					
1000 psi stress	91	100	—	68	43
2000 psi stress	87	100	—	68	45
3000 psi stress	89	100	—	70	48

*Values are in percent of corresponding 90-day values. See Table 6.

TABLE 8—ESTIMATED VALUES OF ATTENUATION PROPERTIES

Condition	Unit weight,* lb per cu ft	Water content, lb per cu ft,† bound in:			Thickness, in., to reduce radiation by a factor of 10‡		
		Free	Cement	Aggregate	Fast neutrons (~2-6 Mev)	Gamma radiation	
						3 Mev	6 Mev
Wet	224.0	5.4	5.1	5.4	8.0	6.9	8.6
Moist§	221.4	2.8	5.1	5.4	8.2	7.0	8.8
Dry	218.6	0.0	5.1	5.4	8.6	7.1	8.9
Heated (85C)	217.4	0.0	4.1	5.2	8.8	7.2	9.0
Heated (200C)	215.7	0.0	2.7	4.9	9.0	7.3	9.1
Heated (350C)	210.5	0.0	2.0	0.4	9.8	7.4	9.2
Dehydrated	~208.1	0.0	0.0	0.0	10.3	7.5	9.3

*Mix data (lb per cu ft): cement, 24.4; mix water, 10.5; coarse aggregate, 101.1; fine aggregate, 88.0.
†Hydrated water retained by cement (w/c): 0.21 at 23 C; 0.17 at 85 C; 0.11 at 200 C, 0.08 at 350 C.
See Table 4 for water retained by hydrous-iron aggregates at various temperatures.
‡Computed values, build-up not included. See References 5 and 6.
§Corresponds to well cured, damp concrete in which about 75 percent of the mix water is still present in the hardened concrete.

the attenuation effectiveness for fast neutrons decreases at a somewhat greater rate with water loss. The neutron attenuation data for concrete heated around 350 C, or higher, must be used with caution since the effectiveness of dried concrete for attenuating intermediate neutrons decreases more rapidly than for fast neutrons.

ACKNOWLEDGMENT

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Weinland Bridge in Switzerland (in Dutch)

P. WARMENHOVEN, *Cement* (Amsterdam), V. 10, No. 21-22, Oct. 1958, pp. 891-892

Reviewed by JOHN W. T. VAN ERP

This is one of the largest prestressed bridges built recently of bold design. Continuous over 4 spans of 186, 250, 288, and 216 ft, respectively. Girders are uniformly 13 ft high, but of varying width, 20 in. at midspan and 38½ in. at supports. Construction was carefully planned and timed to get by with a minimum amount of scaffolding. Work was planned so that most of the creep had taken place before final prestressing. Prestressing was by large BBRV system cables of 125 tons each. Unusual feature was indirect lighting from top rail of aluminum handrailing.

Two-level reinforced concrete bridge in Moscow (in Russian)

V. G. ANDREEV, G. I. ZINGORENKO, and N. N. RUDOMAZIN, *Beton i Zhelezobeton* (Moscow), No. 11, Nov. 1958, pp. 401-410

Reviewed by D. WATSTEIN

A new two-level reinforced concrete arch bridge across the Moscow River is being completed. The portion of the bridge over water consists of a center arch span 354 ft long and two side arch spans each 148 ft long. The outstanding feature of this bridge is the fact that 63 percent of the gross volume of concrete was precast, and 21 percent of it is prestressed concrete.

Another notable feature of this project is the new system of erection of the arches and the stiffening girder which extended along the entire length of the three arch spans at the lower level. The precast segments of the arches and the stiffening girder were assembled on the river bank. After the entire three arch assembly and the transverse framing were made into a monolithic structure by welding of reinforcement and concreting of the joints, the stiffening girder was prestressed. Total force required for the tensioning of the tendons in the center portion of the girder was 4000 tons. The entire assembly was then transferred to a barge and floated into place.

The tendons in the stiffening girder mounted on the outside along the upper and lower flanges of the girder will be left unbonded for 1½ to 2 years. The losses in the tension due to shrinkage and creep will then be corrected and the tendons will be embedded in concrete. During the intervening period the exposed tendons will be protected by a special metal housing.

Postwar bridge construction in Berlin (Berliner Brückenbau in der Nachkriegszeit)

HANNS TOCKUS and KLAUS SCHWITTAY, *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 9, Sept. 1957, pp. 197-202

Reviewed by VALDIS LAPSINS

There were 120 out of a total of 240 bridges of Berlin destroyed during the last war. Western sector of Berlin lost 84 of its 174 bridges. Now there are 63 of the destroyed ones again in use. Sixteen new bridges have been completed; eight are under construction. Article gives a brief résumé of Berlin progress in bridge construction. The article following (pp. 202-209) describes one Dischinger bridge which draws particular interest for its structure and means of construction.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 10, Apr. 1959 (*Proceedings* V. 55). Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Reconstruction of the Voulte Viaduct, first French long railroad bridge of prestressed concrete (La Reconstruction du Viaduc de la Voulte, premier grand Pont Français en beton precontraint)

N. ESQUILLON, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 10, No. 115-116, July-Aug. 1957, pp. 615-639

Reviewed by HENRI PERRIN

The steel superstructure of the railroad viaduct on the Rhone River, France, was destroyed in 1944 and has been replaced by a prestressed concrete structure. The four old piers, 200 ft center to center, were reused as foundations for frames with inclined legs. The construction, illustrated by many pictures, proceeded symmetrically from pier to pier, by sections cast in sliding forms, hung from a temporary truss bridge.

A new prestressing device was developed for this viaduct: the wires are maintained around a cylindrical spring and anchored by cold formed heads bearing against an end plate. The prestressing is done by rotating the end plate (Boussiron system).

Construction

Concrete hangar with special prestressed concrete features (Le hangar pour avions au Findel, Luxembourg)

EMILE NENNING, *La Technique Modern- Construction* (Paris), V. 13, No. 7, July 1958, pp. 193-199

Reviewed by ALEXANDER M. TURITZIN

The hangar is designed for the maintenance and repair of airplanes. Its 55 m concrete arch roof is supported 10 m above the ground on two-story lean-tos. Lean-tos were planned to serve as workshops, storerooms, and office areas. Concrete was chosen over structural steel because of its superior insulating value, its safe behavior under fire, easy maintenance and adaptability to architectural treatment.

The most interesting features of the design was the flat roof of lean-tos acting as horizontal beams in the plane of the roof. The outside faces of these beams, which would be normally under severe tension due to the thrust of the arch, were prestressed so that no tension would exist under the usual condition of roof loading. The ends of one horizontal beam were tied to the ends of the other horizontal beam, located at the opposite side of the arch, by cables hidden in concrete tie beams placed in the front and the rear of the building just above the hanger doors. In this way the horizontal thrust was equilized by the two outside tie rods, leaving an unobstructed area inside the hangar. The article describes each stage of construction, the sequence and the method of prestressing.

Unusual concrete dome structure

International Civil Engineer and Contractor (London), V. 10, No. 6, Nov.-Dec. 1958, pp. 54-56

Reviewed by F. F. GAULD

Describes concisely the design and construction of a doubly curved paraboloid shell spanning 175 ft over tennis courts at Wimbledon, England. Prestressed tie beams were used, the building being supported on pile groups at the four corners.

Multistory construction in concrete

JOHN A. DERRINGTON, *Proceedings*, Institution of Civil Engineers (London), V. 9 (Session 1957-58), Apr. 1958, pp. 433-458

Reviewed by ARON L. MIRSKY

Interesting survey of recent British practices in design and construction of multistory concrete frames. The use of precasting, prestressing, and flat slab types is covered in some detail.

Town wharf at Provincetown, Massachusetts

PAUL S. CRANDALL, *Journal*, Boston Society of Civil Engineers, V. 45, No. 3, July 1958, pp. 249-254

Reviewed by ARON L. MIRSKY

Pier designed to handle small craft (3000 ton excursion boats and fishing craft) utilizes a reinforced concrete deck (precast pile caps and stringers, cast-in-place slab) on treated timber piles to fulfill requirements of low first cost, low maintenance, and reasonably long life expectancy.

Design and testing of a reinforced concrete chimney of 110 m height (in Dutch)

J. M. LAZONDER and A. C. VERHOEVEN, *Cement* (Amsterdam), V. 10, No. 19-20, Aug. 1958, pp. 764-768

Reviewed by JOHN W. T. VAN ERP

Chimney of a cement plant designed on the basis of experience with older chimney of conventional construction, i.e., reinforced concrete outer shell, acid resistant brick lining, and insulation in between (vermiculite

or slag). System proved unsatisfactory due to leaking of the brick lining, and condensation into insulation. In new chimney, therefore, a ventilated cavity was left between the brick lining and the concrete shell, no insulation. This worked quite well and temperature observations over long periods and under varying conditions proved only a small temperature gradient in the concrete (i.e., small heat loss).

Reconstruction of Whitby Fish Quay

GERALD W. ROOKE and FRANK L. TERRETT, *Proceedings*, Institution of Civil Engineers (London), V. 9 (Session 1957-58), Feb. 1958, pp. 129-144 (discussion Aug. 1958, pp. 569-579)
Reviewed by ARON L. MIRSKY

Former timber structure, on the River Esk in Yorkshire, was replaced by a reinforced concrete slab of 2 ft constant thickness, supported on box bearing piles of steel. Timber formwork required to carry the weight of 2 ft of fresh concrete was naturally heavy.

To stabilize a portion of the old masonry harbor wall, a deep mass concrete toe was constructed.

Racetrack for automobile testing (Bau einer Schnellfahrbahn zur Prüfung von Automobilen)

WOLFGANG ZERNA, *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 11, Nov. 1957, pp. 261-268
Reviewed by VALDIS LAPSINS

Discusses the construction of an oval test track of 330 m width and 720 m length to be used for testing automobiles. Track was erected by Volkswagen firm in Wolfsburg.

Construction Techniques

New method of concrete mixing by continuous automatic process

R. T. RICHARDS, *Reinforced Concrete Review* (London), V. 4, No. 6, 1957, pp. 403-412. *Building Science Abstracts*, V. 31, No. 1, Jan. 1958, pp. 9-10
HIGHWAY RESEARCH ABSTRACTS Nov. 1958

In the continuous mixing plant described, the operator presets three controls: mix, water, and quality; adjusts the cement feed if necessary; and starts and stops the flow by push button. The conveyor belt carries three or four sizes of aggregate plus the cement, accurately and uniformly blended by being fed separately onto the belt. The cement content is controlled by varying the belt speed and the supply of aggregate by means of a control gate to the hopper outlet. The aggregate flows at constant rate and grading; selection of aggregate size is made with a switch.

Weight saving by the use of fiber tubes (in Dutch)

J. J. L. BUISMAN, *Cement* (Amsterdam) V. 10, No. 17-18, June 1958, pp. 711-722
Reviewed by JOHN W. T. VAN ERP

Slab construction with the use of fiber tubes for voids is described. Theoretical derivation of design formula is given and illustrated by examples. Tables and charts for design, also for two-way slabs which are possible provided the long-short span ratio is at least 1.5. Torsional moments in the corners of these slabs are taken care of by a special arrangement of the tubes. Comparative designs of solid and hollow slabs demonstrate economy of the method.

Dams

Prestressed dam (in Japanese)

TOKUTARO SAKANISHI *et al.*, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 43, No. 9, Sept. 1958, pp. 1-10
Reviewed by KIYOSHI OKADA

Part of the auxiliary dam was vertically prestressed to 260 ton per m of crest length by bundles of prestressing wires, 2.9 mm in diameter. The tendons were anchored approximately 8 m deep in andesite bedrock.

Thin arch dam on the Gage River (La Voûte très mince du barrage du Gage)

J. P. FREY and P. CHAUVET, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 10, No. 114, June 1957, pp. 539-560
Reviewed by HENRI PERRIN

The French nationalized electric company, taking advantage of the good local conditions on the Gage River, France, decided to erect a 115 ft high and 470 ft long experimental dam. The thickness varies from 12 to 8 ft. The first part deals with the design methods and the data obtained from numerous measurements. The behavior of the structure and its foundation was carefully recorded under loading and unloading conditions. The dam, in spite of a few cracks, seems to have

reached a state of equilibrium and remains quite unaffected by seasonal temperature changes. In the last part the construction, one of the fastest on record, is described at length.

Construction of Ogochi Dam (in Japanese)

SHIRO SATO, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 43, No. 12, Dec. 1958, pp. 1-23
Reviewed by KIYOSHI OKADA

Reports the details of design and construction of Ogochi Dam, the largest gravity dam now in Japan. Capacity of the reservoir is 19 billion cu m, and the dam is 149 m high and 353 m wide. It was completed in November 1957 about 20 years after the work began.

Design

Behavior of thin plates at the corners (Über das Verhalten dünner Platten in den Eckpunkten)

G. RUMPEL, *Der Bauingenieur* (Berlin), V. 33, No. 2, Feb. 1958, pp. 50-54
Reviewed by ARON L. MIRSKY

Analytical proof, based on considerations of equilibrium and deformation, that twisting moments must vanish at corners of nonrectangular plates and (in certain cases, depending on the boundary conditions for the two edges meeting at the given corner) of rectangular plates, so long as there are no impressed moments acting.

Analysis of certain interconnected arch systems

A. W. HENDRY and L. G. JAEGER, *The Structural Engineer* (London), V. 35, No. 7, July 1957, pp. 256-260
AUTHOR'S SUMMARY

This paper describes a new and simple method for the analysis of two pinned sinusoidal arch ribs. (Since the difference between a sinusoidal and a parabolic profile of the same rise and span is quite small the theory will apply to parabolic arches with reasonable accuracy.) The analysis shows that the bending moment diagram for an arch rib of this type is obtained by subtracting from the simple span bending moment diagram its first harmonic component.

It is also shown that harmonic components of the loading higher than the first produce no horizontal thrust and that a system of arch ribs of negligible torsional stiffness interconnected by cross girders which transmit only vertical forces may be analyzed as a flat grid, provided that the first harmonic of the simple span bending moment diagram or deflection curve, as the case may be, is omitted. Distribution coefficients for grids have been published previously by the authors. A worked example of the calculation is shown and a confirmatory experiment is described in an appendix.

Substitute frames in the analysis of rigid-jointed structures

E. LIGHTFOOT, *Civil Engineering and Public Works Review* (London), V. 52, Part 1, No. 618, Dec. 1957; Part 2, V. 53, No. 619, Jan. 1958
Reviewed by GUNHARD-AESTIUS ORAVAS

A substitute frame procedure in a successive correction method is applied to rigid multistory structures that possesses certain advantages in practical design of such frames.

Design of interconnected bridge girder

D. S. BROOKS, *Civil Engineering and Public Works Review* (London), V. 53; Part 1, No. 623, May 1958, pp. 535-538; Part 2, No. 624, June 1958
Reviewed by GUNHARD-AESTIUS ORAVAS

A new version of grid analysis is introduced in this paper that makes use of a relaxation procedure somewhat similar but less complex than the method developed by Lazarides for the stress analysis of the Dome of Discovery. The methods of relaxation for grid analysis have great practical utility and have not as yet achieved as common acceptance by the profession as its simplified version in the method of moment distribution for planar frames. Experimental data substantiates the validity of the relaxation analysis.

Introduction to dimensional analysis (in Spanish)

MARIANO HERNANDEZ, Reprint from *Ingenieria* (Mexico City), V. 28, No. 1, Jan. 1958
Reviewed by MANUEL RENASCO, JR.

Presents a basic theory on dimensional analysis. Starting with theories of fundamental units and basic physical magnitudes, equations are established for the solution of problems involving several unknowns by methods of proportional simplification and by solving lineal and homogeneous equations.

New method for the analysis of beams on elastic foundations

A. W. HENDRY, *Civil Engineering and Public Works Review* (London), V. 53; Part 1, No. 621, Mar. 1958, pp. 297-299; Part 2, No. 622, Apr. 1958, pp. 444-446
Reviewed by GUNHARD-AESTIUS ORAVAS

A method of characteristic functions analogous to Inglis' basic harmonic functions in vibration analysis are introduced with considerable success in the treatment of beams on elastic foundation. Much more than just an interesting tool for foundation analysis.

Influence of an infinite series of equal circular holes on the bending of thin plates (in German)

O. TAMATE, *Zeitschrift fur Angewandte Mathematik und Mechanik* (Berlin), V. 37, No. 11-12, Nov.-Dec. 1957, pp. 431-442
Reviewed by J. J. POLIVKA

Stress distribution and concentration along a hole in a plate have been investigated previously by various theoretical methods and experimental stress analysis, especially photoelasticity. In this paper Kirchhoff's theory of bending and the method of Muskhelishvili are used for approximate determination of the influence which infinite series of equal circular holes in the plate will have on the stress distribution if the plate is subjected to bending moments. Coefficients of the deflection function are determined by perturbation theory. Some special cases are discussed in numerical examples.

Theoretical study of strain variations in heterogeneous bodies, application to concrete; use of grids for the study of strains (in French)

M. P. DANTU, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 11, No. 121, Jan. 1958, Part 1, pp. 54-77, Part 2, pp. 78-98
AUTHOR'S SUMMARY

Part 1 presents a theoretical study which permits the establishment, by statistical considerations, of formulas giving the average stresses near any point for each of the media of an heterogeneous body (e.g., in the mortar and in the gravel stones for a concrete). These formulas may be extended to the case of variations of temperature and of shrinkage.

The relation between the modulus of elasticity of an heterogeneous body and that of its components is studied theoretically and experimentally.

Actual stresses were measured at a small scale in the gravel stones and in the mortar, and on the sawed face of a concrete prism with gages and photoelastic lacquer. These measurements were repeated on a crude face of concrete. The divergence in relation to the macroscopic stresses is considerable.

Part 2 discusses the use of grids for the study of strains. The object of the method is to give a two- or three-dimensional continuous extensometry for extensions above 0.00021, i.e., in the range of plasticity and creep.

To obtain these results, the "moiré" effect produced by superimposition of a network printed on the test piece and of a control network is used. The spacing of the lines of the networks varies from 0.0039 to 0.00078 in.

The method has been applied to the deformation of metals and of plastics. Since the network after photographic impression may be acid etched, the method is used on metals, even for deformations at high temperatures or under the effect of violent impacts.

Finally, the incorporation of a network in the interior of a block of transparent plastic material allows the application to three-dimensional plasticity and creep. This also permits an alternative to the photoelastic method of frozen stress.

Designing a slab using the fracture line theory

W. J. R. Smith, *Civil Engineering and Public Works Review* (London), V. 53, No. 620, Feb. 1958, pp. 163-167
Reviewed by GUNHARD-AESTIUS ORAVAS

A yield-line analysis study is described for a concrete flat slab in a paper mill in England. Various schemes of fracture patterns are studied and in general help to demonstrate the great utility of the method pioneered by Ingerslev and Johansen.

Designing methods (Bemessungsverfahren)

BENNO LOSER and HELMUT LOSER, Wilhelm Ernst and Son, Berlin, 16th edition, 1958, 351 pp; paper back 22 DM, hard back 26 DM
Reviewed by J. F. LEPPMANN

This classic reinforced concrete design manual has been kept up to date by the son of the late author. In addition to the usual design formulas and tables and a chapter on beam and frame analysis it now covers

such items as ribbed floors with hollow fill blocks of clay tile, lightweight concrete and glass, and a chapter on the special reinforcing steels and wire mesh types used in Germany, Austria, and Czechslovakia. Tables of coefficients for conventional flexural design have been extended to steel stresses of 3.0 and 3.5 metric tons per sq cm (nearly 50,000 psi).

Of interest to engineers who are not used to the metric system will be a section on ultimate flexural design (now officially admitted in East Germany) and a chapter on prestressed concrete design, both by Gottfried Brendel, Dresden. The included tables of coefficients for computing section areas, centroids, section moduli, and kern distances of T-sections, bulb T-sections and box sections are more detailed than those found in the current American literature.

Generously illustrated with tables and numerical solutions of practical problems.

Membrane structures with free edges

J. Pelikan, *Acta Technica* (Budapest), V. 20, No. 3-4, 1958, pp. 275-295

Reviewed by Gunhard-Aestius Oravas

An interesting method of analysis that combines conformal transformation and relaxation into a powerful tool in the stress analysis of membranes exhibiting some free edges. The edge girders that are shaped according to specific space curves prohibit their function as a flexural member. The method should open new possibilities in the ever widening field of shell construction and it demonstrates that in this type of structure there is a lot of room for progress and ingenuity of the designer.

Buckling of a spheroidal shell curved in two directions

Pal Csonka, *Acta Technica* (Budapest), V. 14, No. 3-4, 1956, pp. 425-437

Reviewed by Gunhard-Aestius Oravas

The author develops an approximate buckling formula for spheroidal type of concrete shells that has the following simple form

$$p_{cr} = (0.05)\frac{h^2}{R_1 R_2}$$

where p_{cr} designates the critical surface pressure, h the shell thickness, and R_1, R_2 refer to the two radii of principal lines of curvature. The analysis is based on the actual buckling behavior of a concrete shell built in the winter of 1950.

Materials

Corrosion of iron in general and concrete reinforcing in particular (in French)

R. M. Berthier, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 511, Apr. 1958, pp. 101-105

Reviewed by Phillip L. Melville

Corrosion is considered a risk against which "anti-corrosion" is good insurance. The main aim of the treatment is to make the material passive. In the case of concrete, low solubility salts such as phosphates tricalcic or fluosilicate and fluoaluminate are indicated. Small dosage (1 percent) is rather beneficial to other concrete properties.

Influence of temperature lowering on cement hardening (in French)

L. Blondiau, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 500-508, inclusive; May 1957-Jan. 1958; pp. 141-146, 181-188, 203-218, 290-297, 327-331, 364-368, and 15-32

Reviewed by Phillip L. Melville

The study deals with the effect of temperatures near 0 C on hardening of portland, blast furnace, and sulfate cements. Tests show cements containing slag are more sensitive to low temperatures than portland cements. Cold affects concrete with high water-cement ratios and low quality cements. At 3 C strength losses varied universally with the size of the specimens.

Review of available information on polyvinyl acetate as an admixture for concrete

Clara F. Derrington and Leonard Pepper, *Technical Report* No. 6-486, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1958, 11 pp.

Authors' Summary

Presents a review of available literature pertaining to the incorporation of polyvinyl acetate as a concrete admixture into mortar or concrete mixtures. Polyvinyl acetate has been reported to produce excellent bonding of the fresh mortar to old concrete and to improve several properties of concrete. This review of available literature suggests that mortar containing PVA exhibits improved physical properties when the modified con-

crete is cured in a dry atmosphere. However, when mortar containing PVA is immersed in water or subjected to weather conditions involving high humidities, considerable loss of strength, warping, and cracking result.

Special baritic and strontium cements (in French)

A. Braniski, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 511, Apr. 1958, pp. 108-118

Reviewed by Phillip L. Melville

A detailed study of cements obtained by total or partial replacement of CaO by BaO and SrO. Cements tested were manufactured specially and were of eight types: barium blast furnace, strontium blast furnace, barium slag, strontium slag, strontium high-alumina, strontium-barium high-alumina, barium alumina refractory, barium alumina nonrefractory. Details of manufacture are given. From the investigation, it is concluded that tristrontium and tribarium silicates actually exist.

The special cements feature the following outstanding technological properties:

1. The barium blast furnace and the barium slag cements offer high resistance to sea water.
2. The barium alumina refractory cements show an exceptionally high resistance to fire.
3. The barium slag and barium alumina cement provide a satisfactory shield against x- and gamma rays.
4. The barium alumina cement is not hydraulic.

Synthetic polyelectrolytes as concrete admixtures

G. M. Bruere and J. K. McGowan, *Australian Journal of Applied Science* (Melbourne), V. 9, No. 2, 1958, pp. 127-140

Authors' Summary

It is found that some synthetic polyelectrolytes increase the interparticle attraction and degree of flocculation in cement paste. The effects produced in cement pastes, mortars, and concretes when the interparticle attraction is increased by a representative polyelectrolyte admixture, "Krilium," are described.

When added to cement, Krilium increases bleeding rates and decreases bleeding capacities of cement pastes, reduces flow of pastes and mortars, increases the green strength of mortars, and increases the cohesiveness of mortars.

In concrete, it reduces slump, increases cohesiveness, decreases the amount of bleeding, eliminates aggregate segregation, increases the tolerance to vibration, and does not entrain appreciable amounts of air. Small additions do not decrease compressive strength seriously, provided the concrete is properly compacted.

Possible uses of this admixture to improve concrete mixes and methods of handling concrete are discussed.

Insulating bricks for rotary cement kilns (in French)

J. Comte, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 510, Mar. 1958, pp. 75-82

Reviewed by Phillip L. Melville

Comparative study on the cost of cement production as affected by the use of insulating bricks to reduce heat losses in rotary kilns. The following advantages are anticipated: lower temperature in kiln outer skin; less heat loss; and improved preparation of raw material before entering the clinkering zone. Working zone and exposed area of kiln would be expanded. Other advantages would include: less scaling and dust; reduced weight, torsion, and bending; reduced power loss; higher temperatures; and faster construction at no increase in cost.

Microscopic examination of cement and cement clinker (in Dutch)

K. L. A. van der Leeuw and N. P. van Engelshoven, *Cement* (Amsterdam), V. 9, No. 1-2, Feb. 1957, pp. 49-53

Reviewed by John W. T. Van Erp

Microscopic examination techniques have become important since the twenties. As with minerals, the examination of cement is done primarily with light from above and identification of the various ingredients is only possible on highly polished samples. The cement powder is best examined by embedding in transparent plastic and then polishing. Visually contrasting properties of the various ingredients have to be brought out further by means of chemical etching. Advantage of microscopic examination is the speedily available result (20 min) as against chemical analysis.

Construction materials (Baustoffe)

ALBERT JESUMANN, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 25, Sept. 1, 1958, pp. 1240-1241
Reviewed by ARON L. MIRSKY

One of the series of periodic surveys in *VDI Zeitschrift*, covering recent developments in technology. Included here are brief details of developments in deformed reinforcing bars, prestressing tendons, concrete additives, and cements, all German.

Developments in materials testing equipment are covered in another survey in the same issue, by Alfred Kretschmer (pp. 1242-1243).

Admixtures to concrete (in Dutch)

R. POELS, *Cement* (Amsterdam), V. 10, No. 13-14, Feb. 1958, pp. 559-560
Reviewed by JOHN W. T. VAN ERP

Papers presented at the Congress of the Concrete Products Industry, Wiesbaden, 1957, are: concrete accelerators, retarders, plastifying agents, air-entraining agents, synthetic resins (greater flexural strength), bonding agents, and curing compounds. A complete review of all these materials is given, but the most important item still is the proper proportioning and mixing of the basic ingredients of concrete, and a warning is issued against the excessive use of the admixtures.

Pavements

Concrete road construction, slab dimensions, and joint spacing (in Dutch)

A. J. P. VAN DER BURGH, *Cement* (Amsterdam), V. 9, No. 11-12, Dec. 1957, pp. 474-582
Reviewed by JOHN W. T. VAN ERP

A summary is given of the development of road construction during the last 30 years. The most important item in durability and riding qualities of the road has always been the joint; its construction, spacing, the use of joint fillers, etc.

The most satisfactory system has proved to be to make an expansion joint at the end of each day's work, about 1650 ft apart. The joints are formed by ½ in. thick soft wood plank up to 1¼ in. below top of slab, with a joint filler above. The practice of using dowels has been abandoned as being unsatisfactory as also the use of special supporting slabs under the joints. Concrete slab is 8 in. thick, placed monolithically, no reinforcement. Special care is taken to form a good subbase, mostly cement stabilized sand.

In addition to the expansion joints there are also dummy joints, spaced at 20 ft. These joints are made by a machine which vibrates a felt strip 1/16 in. thick into the soft concrete down to a 2-in. depth.

Riding quality in concrete roads—Investigation of mechanical spreading and finishing

R. H. H. KIRKHAM, *Contractors Record and Municipal Engineering* (London), Apr. 23, 1958, 4 pp.

The Road Research Laboratory in England has spent 7 years conducting tests on methods of concrete road construction using mechanical spreading and finishing and its effect on riding qualities. This report describes some of the tests and the construction methods selected as a result.

Prestressed concrete for airfield pavements

GUNNAR S. PEDERSEN, *Civil Engineering*, V. 28, No. 11, Nov. 1958, pp. 66-69

A brief review of several test sections of prestressed airfield pavements in Europe and a fairly detailed description of the construction at Maison Blanche Airfield in Algiers. The prestressed concrete runway, 8000 ft long and 200 ft wide, and a parallel taxiway 80 ft wide have satisfactorily withstood 4 years of heavy traffic.

Precast Concrete

War-damaged church steeple rebuilt in concrete (in Dutch)

W. VAN DER SCHRIER, *Cement* (Amsterdam), V. 10, No. 19-20, Aug. 1958, pp. 773-780
Reviewed by JOHN W. T. VAN ERP

A wood steeple of an old Gothic church was destroyed during the war and recently rebuilt according to a prize-winning modern design. It was built almost entirely out of prefabricated elements, with concrete left exposed. For maximum durability blast furnace cement was used in a mix of 1 bag per 5.5 cu ft of aggregate. Total weight of the prefabricated elements was 200 tons; they were joined by prestressing and a water repellent silicone treatment was given.

Insulating building blocks (in Dutch)

R. Poels, *Cement* (Amsterdam), V. 10, No. 17-18, June 1958, p. 689

Reviewed by John W. T. Van Erp

These blocks are made from a fireproofed fibrous material of high insulating capacity, developed in Switzerland, with a length of ½ m by a height of ¼ m. Feature is that they are milled by machine top and bottom, so that they are of exact dimensions, permitting them to be stacked rapidly into walls, after which they are filled with concrete. Blocks have an outer skin of dense material and are extensively used for tall building construction.

Precast fireproof concrete panels for a tunnel kiln (in Russian)

G. F. Kusnetcow, *Novaja Tekhnika i Peredowoi Opyt w Stroitel'stve* (Moscow), No. 1, 1958, pp. 6-10

Reviewed by N. G. Zoldners

A tunnel kiln for burning insulating bricks, 78 m long and 2.1 m wide was built from big fireproof concrete panels. Two types of fireproof concrete mixes were used in the precast wall and roof panels. In the zone where temperature is expected to be up to 700 C, both fine and coarse aggregates were made of crushed clay building bricks. In the burning zone, where temperature is expected to be between 700 and 1200 C, both fine and coarse aggregates were made of crushed firebricks with addition of finely ground grog. Normal portland cement (300 kg per cu m) was used in all concrete.

Using precast reinforced concrete sets in the Pioneer Tunnel of Great Northern Railway's Cascade Tunnel, King County, Wash.

E. W. Parsons, *Information Circular* No. 7858, U. S. Bureau of Mines, 1958, 10 pp. (Available from Publications Distribution Section, Bureau of Mines, 4800 Forbes Ave., Pittsburgh 13, Pa.)

Successful replacement of timbers with concrete supports in the Pioneer Tunnel is described. The sets for the tunnel were designed for a superimposed load of 3000 psf on roof slab and 1500 psf on side walls. The sets were placed "skin-to-skin" and thus form a continuous lining. A set as placed is comprised of three precast members and is placed on a continuous concrete footing.

Although concrete sets cost about 50 percent more than conventional wood sets, the report notes that the permanence of the concrete supports will more than offset this difference.

Prefabricated office building (in Dutch)

H. T. Zwiers, *Cement* (Amsterdam), V. 10, No. 13-14, Feb. 1958, pp. 541-544

Reviewed by John W. T. Van Erp

Office building built for a large general contractor was built entirely out of prefabricated elements except the central core of stairways, corridors, and elevator shafts, which provided lateral rigidity for the four-story building. Reasons for using prefabricated elements: shortage of skilled labor, short building period desired, and demonstration of what can be done with concrete in its most perfect form. Most of interior partitions also prefabricated.

Room-size precast concrete wall panels in multistory house construction (in Russian)

E. S. Rainus, *Novaja Tekhnika i Peredowoi Opyt w Stroitel'stve* (Moscow), No. 2, 1957, pp. 5-9

Reviewed by N. G. Zoldners

This article describes the construction techniques used in the erection of five-story 60-unit apartment houses in which the room-sized slag concrete precast wall panels are tied together with the hollow floor slabs and form the actual load bearing structure. The wall plumbing, ventilation ducts, windows, and doors are already installed and the panel surface finished before delivery. All panels and slabs (maximum weight 5 tons) were delivered to the building site from a central plant by specially built panel-carrier trucks. Each element of the structure was lifted into proper place by means of a hugh tower crane. A crew of three men with a crane erected a full story of a house, placing about 120 panels and slabs into proper position in a week.

Use of precast concrete in Leningrad subway tubes (in Russian)

K. A. Kuznetsov, *Beton i Zhelezobeton* (Moscow), Aug. 1958, pp. 293-298

Reviewed by D. Watstein

The experience gained in the construction of about 20 km of Leningrad subway indicates that precast concrete segmental elements can be used in the same manner as conventional cast iron tunnel liners. It is

claimed that economies in the use of metal range up to 85 percent while the cost is reduced by 30 percent.

The tubes have outside diameters of either 18 or 20 ft and consist of ten segmental liners. The minimum thickness of the liners ranges from 2.4 to 3.2 in. The liners are made of a harsh concrete mix having a water-cement ratio of 0.28 to 0.3. The concrete is consolidated by vibration from 4 to 5 min and the forms are stripped immediately.

The paper describes both strength tests and water permeability tests performed on the precast concrete liners. Research is presently in progress to determine the types of coatings and admixtures which might be effective in preventing penetration of water. An attempt is being made to replace welded reinforcement cage of deformed bars with one of malleable cast iron. A study is also being made of the properties of liners of autoclaved sand-lime mixes.

Prestressed Concrete

Wire failure (Draadbreuk)

G. W. P. VAN DER HEIDEN, *Cement* (Amsterdam), V. 10, No. 15-16, Apr. 1958, p. 659
Reviewed by JOHN W. T. VAN ERP

Theoretical analysis shows the speed of a fully prestressed breaking wire to be around 85 ft per sec at the moment deformation energy is transformed into kinetic energy. Proper safeguards should be taken against this danger. In radial structures (tanks) speed of breaking wire has been found to be many times greater than expected. Wires have been known to cut through 1-in. boards.

Partially-prestressed concrete structures (theory and dimensioning) [Teilweise vorgespannte Betonkonstruktionen (Theorie und Bemessung)]

P. W. ABELES, *Der Bauingenieur* (Berlin), V. 33, No. 3, Mar. 1958, pp. 77-84
Reviewed by ARON L. MIRSKY

After a brief history of the idea of partial prestressing, author develops theory and gives seven "comparison examples" (beams of rectangular and I section), to indicate its application and value. While much of material has appeared in English in various British and American publications, this paper is of considerable interest.

Quality control in prestressed concrete production

O. J. MASTERMAN, *Magazine of Concrete Research* (London), V. 10, No. 29, Aug. 1958, pp. 57-62
AUTHOR'S SUMMARY

Outlines a system of quality control applied to prestressed concrete joists in which the control depends on daily testing of a random selection of the actual joists. The nondestructive testing includes obtaining the modulus of elasticity of the concrete and the load at the first crack for each joist; corresponding cubes are made for crushing at 1 and 28 days. Results are analyzed and the mean and standard deviations recorded.

Present span limits for some types of steel reinforced and prestressed concrete structures (Les limites actuelles de portée des ouvrages métalliques, en béton armé et précontraint)

HENRY LOSSIER, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 10, No. 114, June 1957, pp. 519-536
Reviewed by HENRI PERRIN

Author first defines the three factors limiting the span of any structure: the characteristic of the materials (intrinsic limit), their unit cost (economic limit), and their practical dimensions (practical limit).

This method is applied to suspension bridges, arch bridges (steel and concrete), cantilever bridges, prestressed concrete bridges, free standing towers (steel and concrete), and dams.

Thompson-Houston plant in Angers—prestressed self-supporting sheds (l' Usine Thomson-Houston a Angers, sheds précontraints auto-porteurs)

M. T. JEAN-BLOCH, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 10 No. 115-116, July-Aug. 1957, pp. 642-653
Reviewed by HENRI PERRIN

The factory is 574 ft long and 360 ft wide with a clear height of 16 ft 16 in. The roof is made of prestressed concrete barrel vaults, continuous over five bays. The vault span is 28 ft and its average thickness 2½ in. Ribs on the outside face are placed at every column line.

Edge beam and vault are prestressed longitudinally. The construction schedule is presented along with some data on the materials used.

Development of prestressed concrete in Switzerland

P. W. ABELES, *Civil Engineering and Public Works Review* (London), V. 53: Part 1, No. 619, Jan. 1958, pp. 54-56; Part 2, No. 620, Feb. 1958, pp. 183-185
Reviewed by GUNHARD-AESTIUS ORAVAS

Author describes the BBRV method of prestressing prevalent in Switzerland and gives several examples of its structural application including the striking Stigli Bridge by Soutter.

Shear problems in prestressed design (in Dutch)

B. VISSER, *Cement* (Amsterdam), V. 10, No. 15-16, Apr. 1958, p. 609
Reviewed by JOHN W. T. VAN ERP

Theoretical analysis has not been able to entirely solve problems of shear design. Many tests on conventional and some on prestressed structures have supplied valuable information and given empirical results, but much more data are needed, particularly where ultimate design methods are to be applied. To realize this international cooperation is recommended.

Field measurement of creep of prestressed concrete beams (in Japanese)

MISAO SUGAHARA, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 43, No. 8, Aug. 1958, pp. 19-28
Reviewed by KIYOSHI OKADA

Describes about 3½ years' measurements of creep in prestressed concrete beams. The beams are of I-section, 1.3 m high and 30 m long, and built about 5.8 m over the river mean water level. Reports that creep strains after the test period are approximately 1.75 times as large as the initial elastic strains due to prestress and/or load, while they are assumed in design to be 2.3 to 2.6 times as large.

Prestressed concrete shell roof (in Dutch)

A. M. HAAS, and J. G. BAAS, *Cement* (Amsterdam), V. 7, No. 7-8, Aug. 1955; V. 10, No. 19-20, Aug. 1958, pp. 799-803
Reviewed by JOHN W. T. VAN ERP

Shells are continuous over two spans of 131 ft, 2¾ in. thick with transverse ribs to prevent buckling, post-tensioned by bent cables (Freyssinet 12 x 0.2 in.). Bays are 40 ft wide, total area about 5.2 acres. Daylighting by double-glazed aluminum windows in strips in the shells. Air conditioning ducts monolithic with the shells at gutter valleys. Expansion and deformation of concrete structure has been thoroughly considered. Exterior walls are entirely independent of structure and have separate foundations.

Composite truss of new type

HOMER M. HADLEY, *Civil Engineering*, V. 28, No. 11, Nov. 1958, pp. 49-51

In this composite truss the bottom chord is a rectangle of prestressed concrete while webs and top chord are of structural steel. The composite unit is 3 ft 6 in. deep center-to-center of chords and 70 ft long. Loading tests are reported. Tests also showed that the heat of welding the web members to the embedded steel sections in the bottom concrete chord caused no serious weakening or loss of bond.

Third international prestressed concrete congress, Berlin 1958 (in German)

Beton- und Stahlbetonbau (Berlin), V. 53, No. 9, Sept. 1958, pp. 221-247
Reviewed by VALDIS LAPSINS

This issue devoted entirely to reports and papers from the Third International Congress for Prestressed Concrete which was held May 5-10, 1958, in Berlin and was attended by 1150 engineers of 42 countries. Papers present discussion on the latest accomplishments in prestressed concrete on international scale.

Prestressed concrete in China (in Dutch)

Cement (Amsterdam), V. 8, No. 21-22, Oct. 1958, pp. 845-848
Reviewed by JOHN W. T. VAN ERP

Industrial expansion of China has called for a great number of factories for prefabrication of concrete elements. A factory near Peking for prestressed and conventional concrete has a yearly production of 160,000 cu yd. Article classes prestressed elements in the following groups:

(a) Building elements
(b) Elements for railroad bridges, railroad ties. Bridge spans have been standardized at 24 and 26 m.
(c) Miscellaneous elements like piles, tubes, portals for high tension lines, etc.

Use of prestressing has increased tremendously since 1956 and by 1962 it is expected that 20-25 percent of all concrete used in China will be prestressed.

Properties of Concrete

Effectiveness of mineral admixtures in preventing excessive expansion of concrete due to alkali-aggregate reaction

ALAN D. BUCK, B. J. HOUSTON, and LEONARD PEPPER, *Technical Report* No. 6-481, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1958, 31 pp.

AUTHORS' SUMMARY

Twenty materials, representing eight different classes of mineral admixtures, were evaluated, using both chemical and mortar-bar test methods, for their effectiveness in preventing excessive expansion of concrete due to alkali-aggregate reaction. It was found that the chemical tests cannot be used with reliance to evaluate effectiveness, and the mortar-bar test method needs improvement to increase its precision. Each of the replacement materials evaluated will prevent excessive expansion if a sufficient quantity is used. Correlations were found between effectiveness and: fineness, dissolved silica, and percentage of alkali retained by reaction product. The eight classes of mineral admixtures represented in the tests were: granulated blast-furnace slags, natural cements, fly ashes, natural volcanic glasses, calcined opaline shales, uncalcined diatomite, uncalcined quartz, and synthetic pure silica glass.

Concretes containing gypsum-cement-pozzolan blends as binders (in Russian)

A. V. VOLZHENSKY and V. I. STAMBULKO, *Beton i Zhelezobeton* (Moscow), No. 10, Oct. 1958, pp. 363-367

Reviewed by D. WATSTEIN

In the interest of securing a more durable gypsum concrete, a study was made of concretes containing a blend of gypsum, portland cement, and a pozzolanic material. The mixes studied contained blended binders consisting of 50-60 percent of gypsum hemihydrate, 20-25 percent of portland cement and 20-25 percent of pozzolanic material.

Authors state that mixes containing the blended binders in the amount of from 300 to 400 kg per cu m, yielded concretes having compressive strengths in the range of 75-100 kg per sq cm (1060-1420 psi) and masonry mortars with compressive strengths of 25-75 kg per sq cm (360-1060 psi). All mixes with a binder content in excess of 200 kg per cu m have withstood 20-25 cycles of freezing and thawing.

Authors point out that the amount of hydraulically active pozzolanic material added to the binder must be sufficiently small to maintain the concentration of calcium oxide in an aqueous solution during the first 2-7 days of hydration below the level of 0.7-0.9 g per l.

Time deformation studies of two expanded shale concretes

GORDON W. BEECROFT, *Proceedings*, Highway Research Board, V. 37, 1958, pp. 90-105

AUTHOR'S SUMMARY

Reports tests on concretes made with shale aggregates produced in Oregon. Lightweight concrete mixes were developed having workability and strength suitable for use in prestressed members. Creep and shrinkage were measured in prisms cast from selected mixes.

By comparison with other test results, time deformations reported for the expanded shales are not particularly higher than those found for sand and gravel concrete of equal strength. However, measured deformations greatly exceeded those provided for in typical recommendations for estimating losses in prestressing steel.

Creep of plain concrete (Das Kriechen unbewehrten Betons)

OTTO WAGNER, *Bulletin* No. 131, Deutscher Ausschuss für Stahlbeton (Berlin), 1958, 74 pp., 20 DM

Reviewed by J. F. LEPPMANN

A comprehensive presentation of all available test results and theories on creep of plain concrete under constant stress and a critical evaluation of their adequacy for predicting either the creep strain or the ratio of creep to elastic strain and the creep-time function. Only the proportionality of creep to the load intensity (within the limit of working stresses, to the percentage of cement paste in the concrete, and to the square of W/C appear to the author as securely established.

More research is needed to determine the influence of the mineralogical composition, the shape and the grading of the aggregate, the chemical composition and the fineness of the cement, the size and shape of the concrete specimen, the mode of compaction,

the age and degree of hardening at the time of loading, and the humidity and temperature of storage before and after loading. An all inclusive mathematical creep-time function has still to be worked out.

Numerous graphs and tabulations and an extensive bibliography accompany the text.

Contribution to the problem of strength (Príspevok k problémom pevnosti betónu)

A. Pavlik, *Stavebnícky Časopis Slovenskú Akadémia Vied* (Bratislava), V. 6, No. 2, Apr. 1958, pp. 97-115
Reviewed by J. Stork

Deals primarily with the "fundamental strength" (*Urfestigkeit*) of concrete which is understood as a stage of concrete loading representing the limit the structure can withstand under given conditions and still perform its given function. This stage is defined by initial structural cracking of concrete. On the basis of ultrasonic investigation of the structure of concrete, relations have been derived between relative longitudinal deformation by ultimate fundamental load and Young's modulus of elasticity. These relations have been confirmed by tests.

The second part deals with problems of flexural strength of concrete members. Attempts are made to explain some phenomena known from bending tests. It is suggested to use "fundamental strength" as a characteristic of concrete.

Lightweight aggregate concrete made with artificial aggregate (Lahké betóny s umelým kamenivom)

F. Kruml, *Stavebnícky Časopis Slovenskú Akadémia Vied* (Bratislava), V. 6, No. 3, June 1958, pp. 146-154
Reviewed by J. Stork

Describes tests of structural properties of lightweight concrete made with *keramzit* (expanded clay), weighing 390-830 kg per cu m, giving concrete having volume weight 500-2000 kg per cu m, and compressive strength of 5-500 kg per sq cm (70-7000 psi). On the basis of numerous tests the author extends the theory of strength of lightweight concretes to "no-fines" concretes as well as compact lightweight concretes. Results have proved that compressive strength of lightweight concrete should be treated on the basis of principles different from those for conventional concrete. Several new relations are given to be used for proportioning of concrete mixes of various volume weights and predetermined compressive strength. Data are shown on tensile strength, bond, and Young's modulus of elasticity of lightweight concrete.

Instructions for composition and fabrication of concrete with desired properties (in German)

Kurt Walz, *Beton- und Stahlbetonbau* (Berlin), V. 53, No. 6, June 1958, pp. 163-169
Reviewed by J. F. Leppmann

Summarizes and explains standard instructions on the selection, storage, handling, measuring and mixing of component materials. Discusses the effects of cement type; W/C; quantity of cement paste; quality, grain size, and grading of aggregate; consistency; workability; compaction; curing; and age of concrete on its strength, weather resistance, permeability, and resistance to chemicals. The influence of W/C in combination with various cement types on the strength of concrete is illustrated by a graph which was developed on the basis of minimum results of numerous test series.

Alkali-aggregate reaction in concrete

Research Report No. 18-C, Highway Research Board, Nov. 1958, 51 pp. $1

More than a dozen contributors took part in this discussion sponsored by the Highway Research Board Committee on Durability of Concrete—Chemical Aspects. It is presented in three parts: one dealing with field experiences; a second with the laboratory tests that have been devised to detect potentially reactive aggregates; and the third with theoretical considerations and present research. These reports and discussions were not meant to cover all phases of the problem nor has the committee attempted to evaluate the data or discussions.

Rheology of concrete (in Russian)

A. A. Pavlik, *Beton i Zhelezobeton* (Moscow), No. 10, Oct. 1958, pp. 372-378
Reviewed by D. Watstein

Presents results of an experimental and analytical study of the various factors affecting the mobility of concrete mixes. Among the factors studied were grading of fine and coarse aggregate, cement factor and amount of extremely fine material (−0.1mm) in the

mix. The properties of the concrete mixes covered by the investigation were the viscosity of the mix (with and without plasticizer), segregation under vibration sustained up to 5 min and the mobility of the concrete measured by a flow apparatus developed by the author.

The author asserts that economies in the use of portland cement can be achieved by improving the grading of the fine and coarse aggregate.

Corrosion of reinforcing steel in concrete in marine atmospheres

LEWIS and COPENHAGEN, South African Council for Scientific and Industrial Research, Pretoria; reprinted from *South African Industrial Chemist*, V. 11, No. 10, Oct. 1957, 14 pp.

Accelerated corrosion tests in a salt spray cabinet using steel reinforced mortar specimens have indicated that factors of considerable significance are: cover to the steel, protective coatings to the mortar, curing, and water-cement ratio of the mortar. The effects of these variables on corrosion prevention are given.

High-pressure test for determining air content of hardened concrete

E. E. McCOY, JR., *Miscellaneous Paper* No. 6-286, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Oct. 1958, 24 pp.

AUTHOR'S SUMMARY

A high-pressure test apparatus and method developed from one originated by the Illinois Division of Highways was studied using 6 x 12-in. concrete cylinders. In this method a specimen is submerged in water in a strong sealed container and subjected to 4000 psi hydraulic pressure. The ensuing apparent volume change is used to determine the amount of air present in the specimen. Results of tests are reproducible and discriminating, agreeing generally with tests by previous investigators and showing excellent agreement with carefully performed micrometric air determinations. The apparatus appears to be suitable for routine tests of large numbers of specimens and can be constructed at a reasonable cost; many tests can be run in one day—easily 12, and possibly 40-50 per day with experienced operators. However, it appears that the high-pressure test might have an appreciable effect on the physical properties of the test specimens, which should be kept in mind where specimens are to be tested subsequently for physical properties other than air content. Construction drawings are included in the report, and a test method is given in an appendix.

Structural Research

Model tests in Portugal

MANUAL ROCHA, *Civil Engineering and Public Works Review* (London) V. 53: Part 1, No. 619, Jan. 1958, pp. 49-53; Part 2, No. 620, Feb. 1958, pp. 179-182

Reviewed by GUNHARD-AESTIUS ORAVAS

A general description of a series of experimental model analyses for wind loaded high structures, arched bridges, dams, turbine spirals, high tension towers, aqueducts, framed structures, skew bridge slabs, ribbed domes, and vibration study of high monument structures.

Simple method of applying electric strain gages to concrete reinforcing bars

GEORGE G. GOBLE, *The Trend in Engineering at the University of Washington*, V. 10, No. 4, Oct. 1958, pp. 11-13, 23

Reviewed by ARON L. MIRSKY

Method described was developed for use in research on ultimate strength of doubly-reinforced concrete beams, requiring about 30 minutes from start of test to failure of beam. It parallels methods previously developed in many laboratories for tests requiring appreciable lengths of time, but is simpler.

Investigations of effect of concrete shrinkage on the gages, methods of compensating for temperature changes, selection and mounting of gages on the concrete in the compression zone of the beams, and detection of type of failure and point of first yield, are also reported.

Bond strength of deformed bars

AXEL EFSEN, *Nordisk Betong* (Stockholm), V. 2, No. 3, 1958, pp. 321-324

Reviewed by MARGARET CORBIN

In previous paper, the author suggested a rating for bond of concrete to deformed reinforcing bars of various types. This paper describes tests conducted at the Laboratory of Applied Mechanics of the Danish Institute of Technology to substantiate this classifica-

tion. The test specimens were spiral splices subjected to pull-out tests, the length of the splice being so short that pull-out and not rupture of the main bar took place. With "W.G" steel accepted as a standard of 100 percent, Kam-Steel was rated as 92 percent, and Tor-Steel as 54 percent.

Reinforcement for autoclaved sand-lime mixes (in Russian)

M. M. Jankauskas, *Beton i Zhelezobeton* (Moscow), No. 9, Sept. 1958, pp. 341-343
Reviewed by D. Watstein

Describes bond tests made with beams and pull-out specimens of autoclaved sand-lime mixes. Three types of reinforcement were included: plain bars, indented bars, and hot-rolled deformed bars. Compressive strengths of the sand-lime mixes ranged from 1600 to 3100 psi, and ratios of reinforcement ranged from 0.42 to 1.35 percent.

Concludes that the load carrying capacity of beams made of autoclaved sand-lime mixes cannot be fully developed with plain or indented reinforcing bars. Recommends that only hot-rolled deformed bars be used in beams of this type.

Fatigue limit of reinforced concrete under repetitive loads (in Russian)

A. I. Ivanov-Diatlov, *Beton i Zhelezobeton* (Moscow), No. 9, Sept. 1958, pp. 353-356
Reviewed by D. Watstein

Author carried out fatigue and static load tests at the Moscow Highway Institute to determine effect of pulsating loads on the propagation of cracks, change in the flexural rigidity, and the flexural strength of beams. The specimens were beams 8 x 20 cm in cross section and had a span of 100 cm. The pulsating load was applied at a rate of 500 cycles per min.

The tests showed that web reinforcement increases the fatigue limit of reinforced concrete beams. Beams containing bars 6.5 mm in diameter sustained 2,000,000 cycles of a pulsating load equal to 65 percent of the ultimate static load, while beams with 19 mm bars sustained a pulsating load equal to 60 percent of the ultimate static load.

The static strengths of beams which had sustained 2,000,000 cycles of a pulsating load were nearly the same as the static strengths of beams which had not been subjected to a pulsating load, and sometimes even exceeded them. It was also noted that web reinforcement, consisting of welded wire fabric or stirrups welded to longitudinal reinforcement, may give rise to a brittle failure at welds.

General

Concrete structures at the Brussels World's Fair (in Dutch)

E. Valcke, *Cement* (Amsterdam), V. 9, No. 11-12, Dec. 1957, pp. 434-462
Reviewed by John W. T. Van Erp

Of the many interesting structures the most spectacular are described: the 1400 ft long street car tunnel, the 1300 ft long viaduct in prestressed concrete, and the civil engineering pavilion. Part of the latter is an extremely daring construction of a footbridge over a ½ acre relief map of Belgium. The footbridge is a 3½ in. slab suspended by cables from a gigantic concrete cantilever arm 80 ft above the footbridge. This girder, an inverted A cross section, cantilevers out over 264 ft. It tapers down toward its cantilevered end so that it is of uniform strength. Stresses are the same in each cross section. Wall thickness of the A-section tapers from only 1½ in. to 5¼ in.

Concrete gasholders (in Dutch)

Cement (Amsterdam), V. 10, No. 21-22, Oct. 1958, pp. 897-900
Reviewed by John W. T. Van Erp

Article is based on the German book by Petermann, "Gasbehaelter in Stahlbeton." Most used system is the storage of gas under a horizontal disk sliding up and down in cylindrical tank. Critical point is tightness of the joints; various systems are described. Construction systems for tank wall (slip-forms, prefabricated elements) foundations, etc., are discussed, also system for heating of tank wall.

Concrete pocketbook 1958 (Betonkalender 1958)

Parts 1 and 2, Wilhelm Ernst and Son, Berlin
Reviewed by John W. T. Van Erp

The yearbook, still in handy pocketsize, becomes more extensive every year. In addition to its usual contents about design and construction, the first part contains as

new additions: tables for uniformly loaded rectangular plates which permit a rapid but more accurate determination of the moment distribution throughout the entire slab area.

There is a chapter on an abbreviated method of moment distribution and a chapter on composite beam construction, showing a great variety of shear connectors. The second part shows even more changes, like the chapters on: prefabricated elements, construction methods, road construction, and walls in tall building construction.

Autogenous pressure welding, a new welding method for concrete reinforcement (Die autogene Preszschweiszung ein neues Schweiszverfahren für Betonstahl)

W. Arnds and J. Bonzel, *Der Bauingenieur* (Berlin), V. 33, No. 4, Apr. 1958, pp. 138-144

Reviewed by Aron L. Mirsky

The history of pressure welding, descriptions of commercial apparatus for pressure welding reinforcing bars, and results of various tests of the welds are given.

Can the simplicity and ease of use of the apparatus, and the high quality of the joints, portend the ultimate elimination of bar lapping, with all its attendant ills?

Swimming pool—aquarium (in Dutch)

Cement (Amsterdam), V. 10, No. 21-22, Oct. 1958, pp. 877-878

Reviewed by John W. T. Van Erp

This structure at the trade fair in Milan, Italy, is intended to be used for the study of human behavior under water: frogmen, divers, etc. It is a tank 27 ft x 13 ft x 10 ft with glass paneled walls set in a concrete frame. Bottom is also of glass panels for observation from below. Glass is double plate with a sheet of plastic in between (safety glass). For best visibility columns are kept slender.

Cement concrete in art (in Dutch)

H. op de Laak, *Cement* (Amsterdam), V. 9, No. 1-2, Feb. 1957, pp. 24-29

Reviewed by John W. T. Van Erp

Describes and illustrates numerous ways in which concrete has found application for art and decorative purposes. Various methods are described in detail such as concrete relief work used for walls and panels, exterior or interior. The decoration is applied on the inside of the forms and thus becomes part of the concrete panel. Another technique is glass in concrete; concrete panels in which glass blocks or fragments of heavy glass are imbedded so that a certain ornamental effect is obtained. Striking effects are possible through the use of colored glass. These panels are used for exterior walls. A technique for interior panels or walls is so called Sgraffito. Concrete consists of layers of different colors. By scratching certain thicknesses away colors are exposed and the desired color pattern is achieved.

Welding of high strength steel (in Dutch)

G. Smit, *Cement* (Amsterdam), V. 10, No. 13-14, Feb. 1958, pp. 563-566

Reviewed by John W. T. Van Erp

This high strength steel (Torstahl) is cold deformed and its greatest strength improvement is at the surface. Welding is quite possible but only permissible by resistance buttwelding, and under strictly controlled conditions. Local building departments always require special permits for it. Describes details of welding procedure.

Masters of modern architecture

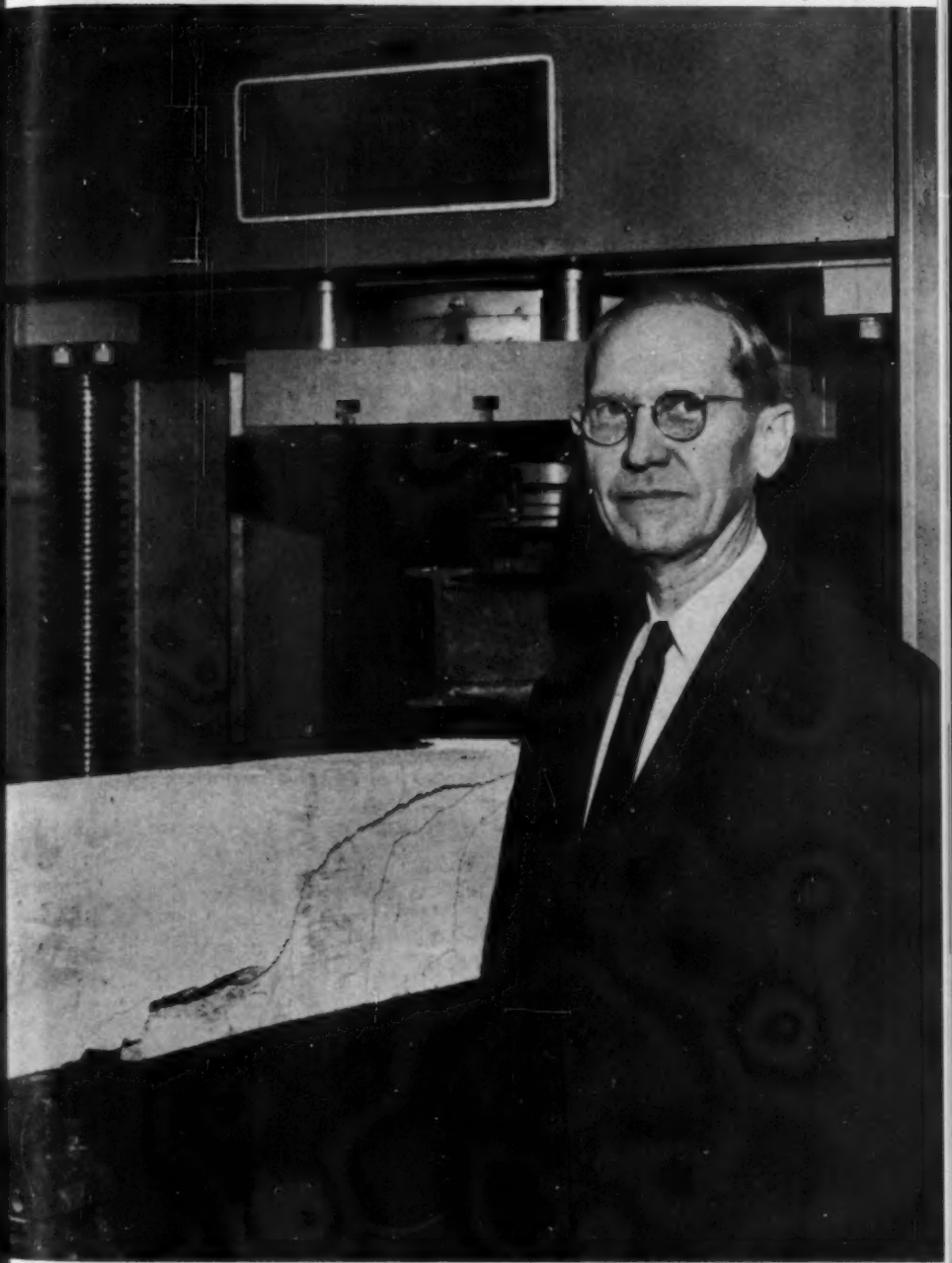
John Peter, George Braziller, Inc., New York, 1958, 230 pp., $15

An art book of modern architecture. A "strong" and interesting introduction leads into a photographic display of the work of more than 60 architects. Many of the structures have used concrete in interesting, economical, and structurally sound ways (for example the works of Le Corbusier, Maillart, Nervi, Torroja, and Niemeyer). There are also some notable omissions such as the work of Candela, Stone, and others.

Although the photographs are outstanding and the layout "arty" there is no logical arrangement of illustrations; that is, the work of various architects is not kept together nor as a second choice are the projects arranged chronologically or by type of structure. Descriptive captions appear not with the pictures themselves (where they would have been most helpful) but in an appendix. It is somewhat disappointing also that the descriptions are so brief, since in many instances interesting architectural and structural details could have been added with only small extension of the text.

ACI News Letter

ol. 30, No. 10 JOURNAL of the AMERICAN CONCRETE INSTITUTE April 1959



ACI President Phil M. Ferguson

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News Letter Contents

April 1959

On the Cover—Incoming President Phil M. Ferguson, professor of civil engineering, University of Texas, was installed at the 55th annual convention in Los Angeles. Full story of the convention begins on p. 3.

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Convention highlights:

Standout Program Presented at 55th ACI Meeting

Los Angeles played host to over 600 ACI members who gathered at the Statler Hilton Hotel February 23-26 to attend the 55th ACI annual meeting.

An outstanding program of technical papers and reports on the latest techniques, design, and research attracted leaders in concrete and related construction fields from all over the world.

February 23 and 24 were devoted to technical committee meetings with the first general session on ACI standards, committee reports, and revision of Institute Bylaws convening Tuesday afternoon, February 24.

The formal technical program began Wednesday morning, February 25, with two concurrent sessions. There were two concurrent sessions Wednesday afternoon and Thursday morning with California authors and speakers playing a prominent part. The annual research session followed bringing latest reports of concrete research in progress at institutions throughout the nation.

Highlights of the convention included introduction of ACI officers for 1959; annual awards and honorary memberships presented at the Awards Luncheon along with the presentation of student competition awards; the 47-booth exhibit with both educational and commercial displays; and a "special events" program arranged by the local committee members.

Technical committee meetings

More than 30 technical committees and subcommittees held working sessions during the convention. A day and a half was devoted exclusively to this important part of Institute work.

Committees meeting were: Committee 115, Research, executive group; Committee 116, Nomenclature; Committee 201, Durability of Concrete

in Service; Committee 207, Properties of Mass Concrete; Committee 208, Bond Stress; Committee 209, Volume Changes and Plastic Flow in Concrete; and Committee 212, Admixtures.

Four subcommittees of Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, conducted meetings: Subcommittee 1, Structural Lightweight Concrete; Subcommittee 2, Lightweight Concrete Masonry; Subcommittee 3, Insulating Concrete; and Subcommittee 4, Lightweight Concrete Aggregates. The full committee also met.

Other committees meeting were: Committee 214, Evaluation of Results of Strength Tests of Concrete; Committee 215, Fatigue of Concrete; Committee 216, Fireproofing or Fire Protection of Structures; Committee 314, Rigid Frames for Buildings and Bridges. Committee 318, Standard Building Code, met and various subcommittees of Committee 318: Subcommittee 2, Materials and Tests; Subcommittee 3, Concrete Quality, Mixing, and Placing; Subcommittee 6, General Considerations: Flexural Computations; Subcommittee 14, Prestressed Concrete; and Subcommittee 15, Ultimate Strength Method.

Also Committee 326 (joint ACI-ASCE), Shear and Diagonal Tension; Committee 331, Structures of Concrete Masonry Units; Committee 332, Recommended Practice for Residential Concrete Work; Committee 336, Combined Footings; Committee 339, Allowable Stresses in Reinforcement; and Committee 401, Specifications for Structural Concrete.

Rounding out the committees conducting "working sessions" were: Committee 609, Compaction of Concrete by Mechanical Means; Committee 611, Inspection of Concrete; Committee 622, Formwork for Concrete; Committee 623, Cellular Concretes; Committee 711, Precast Floor Systems for Houses; Committee 712 (joint ACI-ASCE), Precast Structural Concrete Design and Construction; Committee 716, High-Pressure Steam Curing; and Committee 717, Practice in Low-Pressure Steam Curing.

A meeting was also held for ACI chapter representatives.

Bylaws Changes Approved

The annual business session was held Tuesday afternoon, February 24. President Douglas McHenry presided at this meeting which voted to submit a change of Institute Bylaws to the membership. Also approved, were two new ACI Standards, "Recommended Practice for Hot Weather Concreting," reported by Committee 605, and "Recommended Practice for Proportioning Lightweight Aggregate Structural Concrete," reported by Committee 613. A revision of "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)" was also approved. The new and revised standards will now go to the membership via letter ballot. A progress report of the joint ACI-ASCE Committee 326, Shear and Diagonal Tension, was also included in the meeting.

The convention was welcomed to Los Angeles by Col. Carroll T. Newton, district engineer, U. S. Army Corps of Engineers, Los Angeles District.

ACI Bylaws changes

A petition to revise the Institute Bylaws to provide for an increase in member dues was approved for referral to the ACI membership for secret letter ballot.

ACI dues have been maintained at the same level since 1951. In the interim there have been many economic changes. The most important to an organization on a fixed per capita income from members is that of creeping inflation with constantly rising costs. Each ACI member is now getting services which cost almost $26 per year for which he is paying only $15.

A year's study of the Institute's financial status by a special committee of the Board of

Direction resulted in the petition to revise the Bylaws to provide for an increase in member dues. The Board of Direction agreed that curtailment of services or even maintenance of the *status quo* did not meet the desires of the majority of members or the solution to the impending financial pinch; rather the consensus was for more not less service to members.

The proposed changes to Article IV of the ACI Bylaws change the dues structure as follows: Contributing Members $135 (previously $100); Corporation Members $65 (previously $50); Members (individuals) in North America and U. S. Possessions $20 (previously $15); Members elsewhere $16 (previously $12); and Junior Members $10 (previously $7.50). The dues for Student Members remains at $5. The complete wording for Article IV can be found in the News Letter section of the December, 1958, JOURNAL, p. 5.

Two ACI regulars were honored at the 55th convention. Douglas E. Parsons (left) received the Henry C. Turner Award and H. J. Gilkey was made an Honorary Member of the Institute. Eduardo Torroja was also made an Honorary Member but was unable to be present. The late Hardy Cross was similarly honored

Report of ACI-ASCE Committee 326

A progress report of the work of ACI-ASCE Committee 326, Shear and Diagonal Tension, was presented by Richard C. Elstner, Portland Cement Association, Skokie, Ill. The report pointed out that diagonal tension is dependent on two quantities, shear and flexure. Present design methods consider shear alone with the effects of flexure considered sufficiently protected by the factor of safety. Recent laboratory tests indicate that the effects of flexure can be important and under certain normal design conditions this effect can reduce the margin of safety built into present design procedures. This condition can be corrected either of two ways.

The first alternative is to continue present procedures but increase the safety factor. To make the present procedure satisfactory for all girders, present allowable nominal shearing stresses must be reduced about one-third. This method would be conservative for the majority of girders to insure proper safety for the worst condition.

The second alternative requires a revision of the fundamental ideas and philosophies of shear and diagonal tension, i.e., to consider both shear and flexure in design equations. When the influence of flexural stress is small, the allowable nominal shearing stress can be high, actually more liberal than the present code. However, when the influence of flexural stress is great, allowable nominal shearing stress must be lower and more conservative than the present code. The committee favors the second alternative.

A task group of Committee 326 has been reviewing test data and proposed equations expressing shear and diagonal tension. The committee is preparing a report on shear and diagonal tension which will include recommended design procedures.

The progress report noted that it was necessary to establish a criterion for the capacity of beams without web reinforcement. Such a criterion could be based on two conditions, cracking load or ultimate load. The committee favors basing the criterion on the former because the difference between cracking load and ultimate load is not a reliable quantity and the effects of sustained load on a diagonally cracked beam are uncertain.

Committee 326 has not limited itself to studies of beams without web reinforcement. Beams with axial load, beams with web reinforcement, slabs, and footings have also been studied. In all cases it can be shown that the conditions on which the present design equations are based differ from actual conditions within the structural member.

While the fundamentals of shear and diagonal tension are not yet fully understood, the point has been reached where it seems necessary to revise the classical procedures because they are based on assumptions which do not apply to present day structural members. Expressions based on rational parameters are in the process of development.

New Standards Expand Scope of ACI Mission

The 55th annual convention of the Institute approved two new standards in fields not specifically covered previously. Convention participants voted to accept committee reports on hot weather concreting and proportioning lightweight structural concrete and present them to the membership for approval as Institute standards. Also approved to be placed on the secret letter ballot was a revision to the 1942 standard "Recommended Practice for Measuring, Mixing, and Placing Concrete."

Hot weather concreting

"Proposed Recommended Practice for Hot Weather Concreting" was reported by ACI Committee 605, Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C., chairman.

This standard provides information useful in minimizing detrimental effects of hot weather on concrete. Means are described for reducing concrete temperature by proper attention to ingredients; methods of production and delivery; and care in placement, protection, and curing. Information is given on the use of admixtures to reduce mixing water requirements and to retard setting. Emphasis is given to the importance of meticulous attention to the use of standard procedures in testing concrete made in hot weather.

The problems of concreting in hot weather have been increased in recent years by a number of factors, including more finely ground and more rapidly hardening cements, handling of larger batches of concrete, use of thinner sections, and increased speeds in all construction operations. This standard suggests procedures for reducing the hazards of hot weather in the more usual classes of construction.

Proposed standard covers elements of importance in controlling temperatures of concrete including cooling of cement, use of admixtures to retard hardening, production and delivery, placement and finishing, protection and curing, and temperature records. Specific methods for accomplishment of these objectives are outlined.

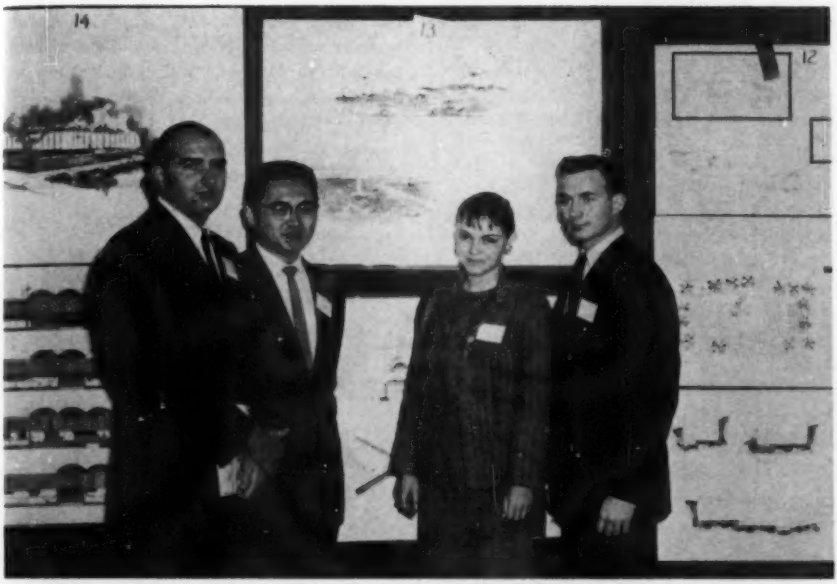
Lightweight concrete proportioning

"Proposed Recommended Practice for Proportioning of Lightweight Aggregate Structural Concrete" is part of the work of ACI Committee 613. The subcommittee on proportioning lightweight aggregate concrete, J. J. Shideler, manager, Products and Applications Development Section, Portland Cement Association, Skokie, Ill., chairman, presented the report.

This standard is intended as a supplement to "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" and describes a procedure for proportioning structural grade concrete containing lightweight aggregates. This procedure does not require the use of values for specific gravity or absorption of the aggregates but utilizes a "specific gravity factor." Use of this factor is illustrated and examples are included for proportioning both air-entrained and non-air-entrained mixes.

Measuring, mixing, and placing

"Proposed Revision of Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)" was prepared by ACI Committee 614 under the chairmanship of Lewis H. Tuthill, concrete engineer, Division



Winners of the student competition (l. to r.) Norman Lacayo, second prize, Yin Li, first prize, and Edna Dubnoff and Gene Zellmer, honorable mentions. All are architectural students at the University of California and were selected from 25 entries

of Design and Construction, California Department of Water Resources, Sacramento.

In making these recommended revisions, the committee was guided by the belief that progress in improvement of concrete construction practices will be better served by endorsing a comparatively high standard of practice rather than "common practices;" recommendations are therefore made with the thought of permitting the user flexibility in his specifications to the extent he considers worthwhile for his particular project. Many of the practices recommended are included primarily to improve concrete uniformity. Where the methods described have been carried out properly, the effort and investment have usually been rewarded by concrete of higher quality, a smoother operation, and higher production rates, all factors which tend to offset any additional cost.

Since the main objective of this recommendation is maximum uniformity, homogeneity, and quality of concrete in place, special consideration is given to practices which have been found to contribute most to this end. To portray more clearly certain of the principles involved in these practices, illustrations of good and bad practice are included as a part of this recommendation.

Considerable discussion was presented on the pros and cons of portions of the recommended practice concerned with finish screening. Discussion was presented by Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.; Byron P. Weintz, chief of engineering, Consolidated Rock Products Co., Los Angeles; J. E. Gray, engineering director, National Crushed Stone Association, Wash-

Presidential Address

Full text of address delivered by retiring President McHenry at the Awards Luncheon, February 25, commenting on the concrete industry with respect to other fields of scientific endeavor and its own history and future, appears on p. 1069 in this month's JOURNAL.

ington, D. C.; Walter K. Wagner, chief engineer, Albuquerque Gravel Products Co., Albuquerque; E. L. Howard, chief testing engineer, Pacific Cement and Aggregates Inc., San Francisco; D. K. Woodin, senior construction inspector, East Bay Municipal Utility District, Oakland, Calif.; Harry F. Irwin, concrete engineer, Warner Co., Philadelphia; Bailey Tremper, supervising materials and research engineer, California Division of Highways, Sacramento; E. A. Abdun-Nur, consulting engineer, Denver; H. J. Gilkey, professor of theoretical and applied mechanics, Iowa State College, Ames; Jack Lendved, director of engineering, Construction Machinery Division, Chain Belt Co., Milwaukee.

After considerable discussion and the rejection of an amendment by the convention, the proposed revised standard was approved for referral to the ACI membership by secret letter ballot.

Construction Session Features Precast and Pavement Concrete

Lewis H. Tuthill, Division of Design and Construction, California State Department of Water Resources, Sacramento, served as chairman of the Construction Session.

The subject of erecting precast concrete is one of growing interest. The growth of the precast industry has outstripped this phase at times. Among the papers presented were two on that topic.

Highway construction remains a subject of great interest. Attendants at the Construction Session heard papers on preparation of subgrades with cement and a plan for qualifying ready-mixed concrete plants supplying highway work.

Erection of Precast Concrete—J. L. PETERSON, president, J. L. Peterson, Inc., Long Beach, Calif.

The rapid growth and development of the use of precast concrete construction in the United States since the end of World War II has occasioned a considerable amount of improvisation and experimentation in the materials and methods of erection. As can be expected there is a growing amount of standardization developing in these methods and materials. While the process is far from complete, at the present time it has developed far enough to establish certain patterns which are an indication of the eventual standard methods and materials which will be used.

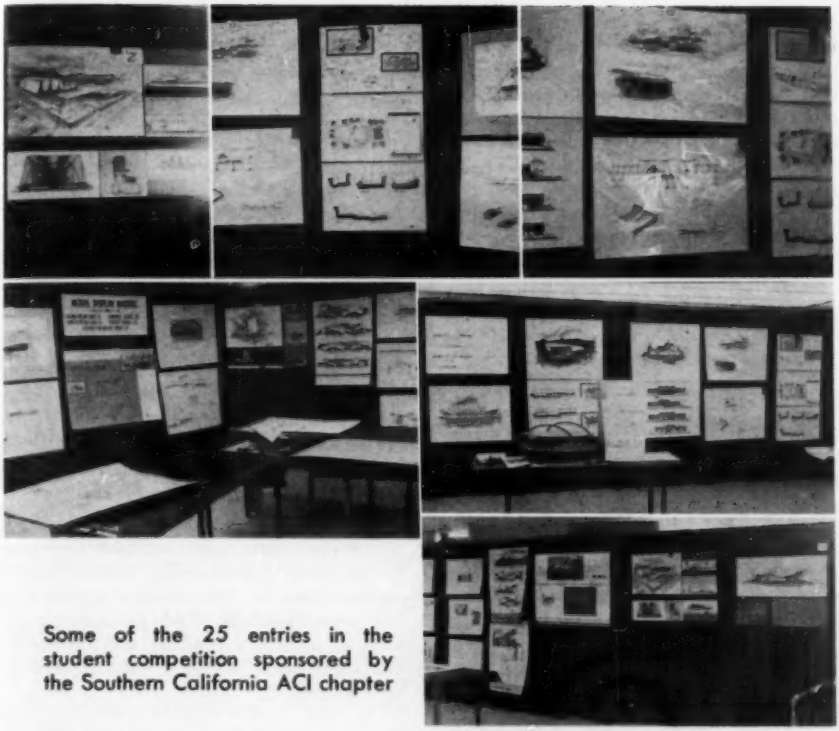
The following subjects were covered briefly: erection hardware; erection of precast concrete walls; erection of precast concrete beams and frames; erection of precast concrete floor and roof slabs; erection of miscellaneous precast concrete units; multiple crane attachments to single precast concrete units; center of gravity method for erection of unsymmetrical units; erection stresses in concrete and steel; and a sample specification for erection of precast concrete.

Joinery of Precast Elements—W. HOWARD GERFEN and JOHN R. ANDERSON, consulting structural engineers, San Marino and Pasadena, Calif., respectively.

The field of precast joinery is so vast that the authors deemed it best to thoroughly cover one subdivision of joinery of precast elements rather than to generalize on the entire subject. Their discussion was confined to methods of joining wall panels.

Seven ways were presented in which precast wall panels may be tied together including three variations of the cast-in-place column, one precast column, one steel column, one wall section, and one welded tie detail.

Authors commented that these details are capable of an infinite number of variations and that contractors are prone to agree on the most economical method of making these connections. A compromise to fit the con-



Some of the 25 entries in the student competition sponsored by the Southern California ACI chapter

tractor's ability, labor force, and equipment to the structural requirements of the job is necessary to arrive at a satisfactory detail.

Cement Treated Subgrades Under Concrete Pavements—J. W. TRASK, assistant state highway engineer, California Division of Highways, Sacramento.

Cement treated subgrade consists of mixing mineral aggregate, cement, and water by road mixing or plant mixing and applying a bituminous curing seal thereto. The purpose in constructing a cement treated subgrade is to get a subgrade that will resist erosion under the pumping action of concrete slabs. In effect the cement treatment is only a means for providing a layer that is firm enough to hold a film of asphalt. In fact, the entire treatment might equally well be described by the term "pre-subsealing." The practice of subsealing concrete pavements has been effective even after pumping and evidence of joint troubles have started. Work has indicated that subsealing before any pumping has been noticed is an effective means of retarding this development. Therefore, when a layer of asphalt is placed over a cement-treated subgrade it is in effect subsealing in advance.

California specifications are summarized and field applications and laboratory research are described.

Qualification Plan for Ready-Mixed Concrete Plants—C. E. PROUDLEY, state materials engineer, North Carolina State Highway Commission, Raleigh. (Report presented by Robert F. Adams.)

Described procedure for rating ready-mixed concrete plants based on plant equipment and layout, materials being used, as well as qualifications of plant employees.

The North Carolina State Highway Commission lists ready-mixed concrete plants approved as sources of concrete for state

highway work on a basis of periodic checks by its engineers and inspectors. A and B ratings of plants on the approved list are based on plant equipment and layout and on qualifications of the plant employees. Plants may be removed from the approved list if their equipment becomes unacceptable or if their services cause an unsatisfactory job.

Materials used by the approved plants are inspected and tested. A training program conducted jointly by the highway commission and the industry qualifies plant employees as concrete technicians.

Cast-in-Place Concrete Pipe—L. H. Kristof, Corps of Engineers, Sacramento, Calif.

A report of cast-in-place concrete pipe projects in the West Coast area.

Special Problems in Concrete

Concrete versus nuclear energy was primary among the subjects of this session. Radiation shielding and blast resistance were among the topics. Also a paper offering a method for temperature control of mass concrete was presented. Four papers were presented at this session with Walter H. Price, chief, Engineering Laboratories, U. S. Bureau of Reclamation, Denver, serving as chairman.

Properties of Shielding Concrete—J. O. Henrie, Atomics International, Canoga Park, Calif.

General shielding requirements were discussed with particular emphasis on fast neutron shields. Concrete materials which are effective in meeting the requirements are outlined and the basic principles involved are explained. The concrete mix, type, and source of materials, and strength properties of the shields of four similar nuclear reactors were compared. The concrete setting time and strength effects of adding boron in the form of mineral colemanite and borocalcite and the counter effects of calcium chloride were described.

Vice-President Phil M. Ferguson (left) presents Retiring President Douglas McHenry with a plaque commemorating his service to the Institute

Performance and Design of Special Purpose Blast Resistant Structures—R. A. Williamson, Holmes and Narver, Inc., Los Angeles.

Located on the numerous coral islands of the Eniwetok and Bikini atolls in the Marshall Islands of the South Pacific are a large number of massive structures designed to withstand the blast, radiation, and other effects accompanying nuclear detonations. Most of these structures are scientific stations, housing measuring and support equipment involved in obtaining information sought by scientists in studying fission and fusion phenomena.

These scientific stations probably constitute, to date, the largest amount of concrete construction of American origin designed to resist the effects of nuclear blast. The service record of these structures and the approach currently used in their design were the subjects of this paper.

The conventional working stress approach will produce structures having safety factors

CALIFORNIA REGISTRATION RECORDS

California provided the largest group of registrants attending the ACI 55th annual convention.

In addition to the 40 other states and Canada, there were representatives from England, Venezuela, and Cuba.

Following are registrations from the various states and countries:

California	316	Georgia	4
Illinois	51	Iowa	4
New York	30	Kansas	4
Canada	22	Maryland	4
Ohio	20	Minnesota	4
District of Columbia	17	Tennessee	4
Michigan	16	Florida	3
Texas	16	Nebraska	3
Colorado	12	West Virginia	2
Arizona	11	Cuba	1
Wisconsin	11	England	1
Washington	10	Idaho	1
Missouri	8	Louisiana	1
Pennsylvania	7	Maine	1
New Jersey	6	Montana	1
Oklahoma	6	Nevada	1
Indiana	5	New Mexico	1
Massachussetts	5	Oregon	1
Mississippi	5	Venezuela	1
Utah	5		

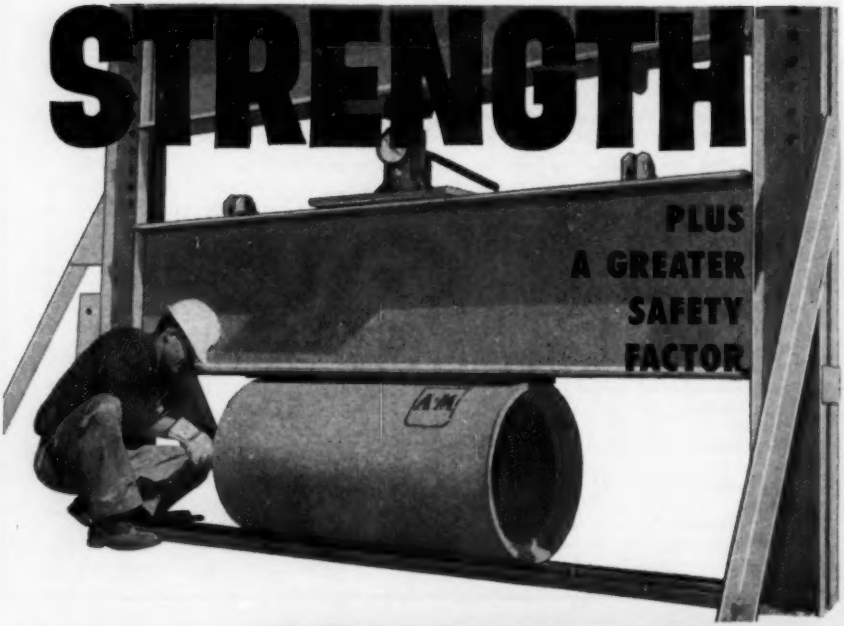
against collapse in the various modes of failure which are not uniform, being too high in some cases and too low in others. The objective of maintaining a consistent amount of reserve strength in each failure mode can only be attained by design procedures which evaluate ultimate strength and consider the transient nature of the loading. The procedures currently used accomplish this goal, involved simple extensions of routine procedures which are easily learned by the average designer, and are adaptable to conditions requiring a maximum amount of output in a short time.

Influence of Grade of Steel on Blast Resistance of Reinforced Concrete Beams—WARREN A. SHAW and J. R. ALLGOOD, structural research engineers, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

Method for determining the influence of grade of reinforcing steel on blast resistance of beams is based on preselected criteria of failure. The peak dynamic load capacity is calculated for beams of a given configuration but having different percentages and grades of reinforcing steel. These calculations are made for blast-type loads of different duration. Plots of the results indicate that suitability of various steel grades will depend primarily on maximum permissible deflection, characteristics of the loading, and amount of tension steel used.

Surface Cooling of Mass Concrete to Prevent Cracking—ROY W. CARLSON, consulting engineer, Berkeley, and DON P. THAYER, California Department of Water Resources, Sacramento.

In the preliminary studies of temperature control for a concrete dam proposed for the Oroville site on the Feather River, Calif., a method was developed which would permit placing of extremely long monoliths without development of excessive tensile stresses from cooling. Although subsequent abandonment of this design for the Oroville Dam has precluded application of this method in this instance, it is believed to be of general applicability. The basic concepts of this method were described, detailed computations of stresses were given, and possible construction methods for accomplishing this temperature control were suggested.



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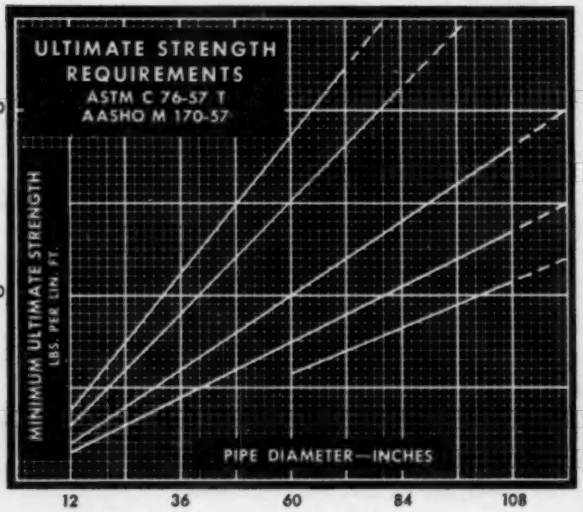

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Continuously Reinforced Pavements Discussed at Design and Analysis Session

There was a variety of topics up for discussion at the Design and Analysis Session. Prestressed lift slabs, and a prestressed shell roof were subjects of two papers. Longitudinal forces on a portal frame for a highway bridge and a report of research on continuously reinforced pavements were also featured. Phil M. Ferguson, Department of Civil Engineering, University of Texas, Austin, was chairman.

Design of Prestressed Concrete Lift Slabs for Deflection Control—EDWARD K. RICE and FELIX KULKA, T. Y. Lin and Associates, Van Nuys, Calif.

Subjects covered include: review of lift-slab construction procedures; moment distribution in a flat slab; present design practice; layout of tendons; design of lifting collars; concrete strength; and special reinforcement at edges and around openings.

Effect of Longitudinal Forces on a Portal Frame Supporting a Highway Bridge Deck—TUNG AU, Civil Engineering Department, Carnegie Institute of Technology, Pittsburgh, and THOMAS D. Y. FOK, Civil Engineering Department, Youngstown University, Youngstown, Ohio.

Longitudinal forces resulting from live load on a highway bridge deck and from bearing friction at supports of bridge girders produce shear, bending, and torsion in the supporting pier. Paper analyzed a type of pier often used in highway bridges, namely the portal frame fixed at supports, subject to such forces.

Loading is assumed unsymmetrical with respect to the center line of the portal frame. Analysis by moment distribution is extended to include the effects of torsion. The advantages and limitations of the method of analysis were discussed, and the solution for frames with prismatic members formulated.

Prestressed Concrete Roofs for Grandstands in Caracas, Venezuela—HENRY M. LAYNE, consulting structural engineer, Los Angeles, and T. Y. LIN, T. Y. Lin and Associates, Van Nuys, Calif.

The new Hipodromo Nacional (National Race Track) now under construction in Caracas, Venezuela, is composed of many buildings and structures required for the complete operation and maintenance of what eventually will become one of the world's largest and most luxurious horse racing establishments.

The immediate concern of this paper was the prestressed concrete thin shell cantilever roofs over the three large five-story grandstands. The prestressed roof is about 100 ft above the ground floor and covers the seating areas. The main cantilever extends 90 ft from supporting columns.

Venezuelan officials preferred that reinforced concrete be used wherever possible and practicable since cement and aggregates were available in the Caracas area. Therefore, the decision was made to carry out the structural design in concrete under the requirements of the ACI Building Code and the Reinforced Concrete Code prepared by the Ministry of Public Works in Venezuela.

Laboratory Research on Pavements Continuously Reinforced with Deformed Bars—J. L. WALING and M. J. GUTZWILLER, Department of Civil Engineering, Purdue University, Lafayette, Ind.

The specimens tested were 28 ft long, 3 ft wide, and 8 in. thick. Either No. 4, No. 5, or No. 6 bars were used as the longitudinal reinforcement with No. 4 bars as the transverse reinforcement. The specimens were cast in a portable form in which the amount of steel, the location of steel, and the depth of slab could be varied. The slabs were cast with preformed cracks at definite locations in the testing region of the slab.

The slabs were tested on an elastic subgrade having a modulus of approximately 160 lb per cu in. and facilities were available to vary the subgrade modulus as desired. The

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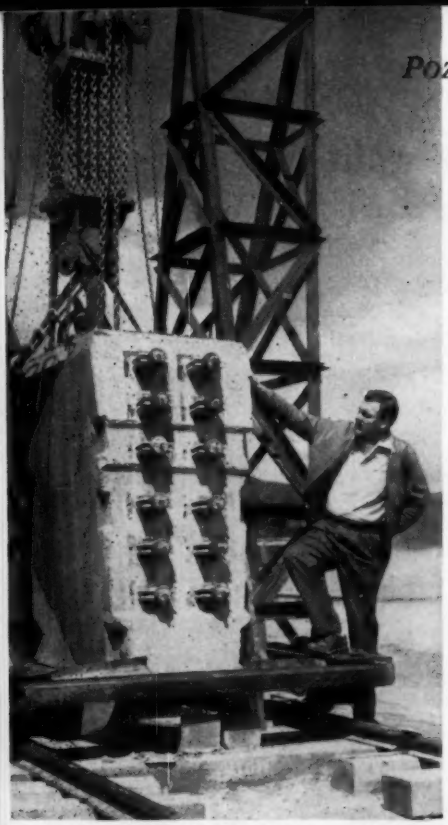
Prestressed concrete beams for 6 bridges at Air Academy



PRECAST BRIDGE BEAM is inspected after post-tensioning. Construction of all prestressed POZZOLITH concrete beams by A. S. Horner Construction Co., Contractors, Denver. Bridges built under supervision of the Air Force Academy Construction Agency.

Part of the striking man-made landscape at the new U. S. Air Force Academy are six major concrete bridges—4 highway and 2 railway spans—that required over 2½ miles of prestressed concrete beams.

POZZOLITH concrete was specified and used for all 128 beams. The mix was designed to produce 4% entrained air, 2-inch average slump and high early strength. Compressive strength of 4500 psi—required before stress application—was achieved in 3 to 5 days with POZZOLITH. Seven day strength was approximately 6500 psi and 28-day strength was well above 7000 psi.

Over 750,000 cubic yards of POZZOLITH concrete were used throughout the Academy complex—to best meet a wide range of concrete requirements. POZZOLITH best provides control over three important concrete quality factors: water content, entrained air and rate of hardening—the *key factors* in obtaining uniform, superior quality concrete.

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LARGEST OF 6 BRIDGES includes five 120-foot spans of prestressed POZZOLITH concrete beams. Bridge deck is 71-feet wide. Architects: Skidmore, Owings & Merrill, Chicago. Engineer: L. Boduroff, Denver. Prestressing: Prescon Corp., Corpus Christi, Texas.

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loads applied to the slabs were vertical static loads to simulate traffic loads and horizontal loads to simulate stresses induced by temperature changes. Stresses in the bars at various locations along the bars were measured by SR-4 strain gages. Vertical deflections of the slab were obtained with Federal dial indicators and surface strains or crack widths were obtained with a Whittemore strain gage.

Stresses and Deflections in Pavements Continuously Reinforced with Deformed Bars—J. L. WALING and M. J. GUTZWILLER, Department of Civil Engineering, Purdue University, Lafayette, Ind.

Results of a series of laboratory experiments on simulated continuously reinforced concrete slabs were given, with those results pertaining to slab deflections, crack widths, and stresses in deformed bar steel reinforcement being emphasized. Some of the significant findings of these laboratory experiments were compared with field observations reported in the literature and several criteria were suggested for optimum structural design of continuously reinforced pavements.

Time out from the convention for (l. to r.) Col. Carroll T. Newton, who welcomed the Institute to Los Angeles, President McHenry, and Paul W. Abeles, visiting from England

Concreting Materials and Methods

Joe W. Kelly, professor of civil engineering, Engineering Materials Laboratory, University of California, Berkeley, was chairman of the Concreting Materials and Methods Session. The subjects discussed were well diversified and interesting. The papers covered evaluation of mortar mixes, properties of shotcrete, tremie concrete control, and the selection of materials for Glen Canyon Dam.

Evaluation of Concrete and Mortar Mixes—W. A. CORDON, Department of Civil Engineering, Utah State University, Logan.

Properly proportioned concrete mixes are designed to produce quality, workability, and economy. Concrete mixes must be proportioned to produce required quality and adequate workability, but to completely evaluate a mix, its efficiency must also be considered.

Data were included which evaluate mixes of varying water content and water-cement ratio, varying aggregate gradings and particle shape, different cements and pozzolans, and the influence of various admixtures on the efficiency of concrete mixes.

Much useful information can be obtained by evaluating concrete mortars. Particle shape and grading of sand has significant influence on the properties of concrete. This is studied by evaluating the mortar from concrete mixes and correlating with concrete tests. New laboratory apparatus used in the study of concrete mortars was discussed.

Shotcrete—Properties and Applications—STANLEY G. ZYNDA, manager and consultant, Gunite Contractors Association, Los Angeles.

A general summary of properties and physical characteristics including water-cement ratio investigation and application to new work.

Although high strengths with shotcrete are obtainable, they are not commonly in the 9000 psi range nor is such strength desirable. It is common to find 4000 psi shotcrete speci-

Looking ahead to

1960 in New York City

ACI's Technical Activities Committee is already shaping up program plans for the 1960 convention to be held in New York City, March 14-17.

Those who intend to offer papers for presentation at the 56th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1959, and final manuscripts of accepted papers will be due Jan. 1, 1960.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1960 program

fied where 2500 psi is adequate for design. In many designs, due consideration is not given to placing procedure and considerable economy can be lost in forming. Heavy forming is unnecessary as there is no hydrostatic pressure to overcome.

There are many advantages and economies to be gained through the use of shotcrete in building work. However, the economy begins with the design and not in the field where often shotcrete is used to do a job found too late to be impractical by other methods.

Controlling Tremie Concrete with Admixtures—J. WAYMAN WILLIAMS, JR., manager of technical service, Sika Chemical Corp., Passaic, N. J.

Noting that nonuniform quality and excessive laitance are disadvantages usually associated with tremie concrete, the paper reported on laboratory tests and field applications which show that retardation and air entrainment will reduce laitance and improve the flow, the uniformity, and the pattern of heat development in tremie concrete.

Concrete and Concrete Material for Glen Canyon Dam—WALTER H. PRICE, L. P. WITTE, and L. C. PORTER; respectively, head Engineering Laboratories; chief, Concrete Laboratory Branch; head, Aggregate and Durability Unit, Concrete Laboratory Branch; U. S. Bureau of Reclamation, Denver, Colo.

Glen Canyon Dam, a concrete arch type, will be approximately 700 ft high and 300 ft thick at maximum section and 1500 ft in length and 35 ft thick at the crest. Approximately 5 million cu yd of concrete will be required in its construction.

This report recounted some of the highlights of the many factors which were considered in the specific field of concrete and concreting materials for Glen Canyon Dam. It pointed out the methods used in solving some of the many and varied problems encountered in the design and construction of a project of this magnitude.

The only deposit of natural aggregates within an economical distance of the dam site which contains sufficient material for construction of the dam and appurtenant works did not meet the usual U. S. Bureau of Reclamation grading requirements for concrete aggregate and in addition contains excessive amounts of lightweight material and other deleterious materials. The aggregate investigations and preliminary mix proportions were described to show how the Bureau's usual specifications were modified to meet the problems at hand and still obtain adequate strength, durability, etc. To accomplish removal of lightweight material, the contractor elected to employ a heavy media flotation process.

The mass concrete for the dam will use a combination of portland cement and pozzolan cementing materials; Type II cement will be used.

Concrete will both be precooled and postcooled. Precooling to assure placing temperatures of 50 F or less will be accomplished by sprinkling the coarse aggregate in route to the batching bins where it will be further cooled by refrigerated air blasts, by cooling the mixing water, and by the addition of chip ice to the mix. Postcooling to control tem-

perature rise resulting from hydration of the cement will be accomplished by means of cool water circulating through a system of pipes embedded in the concrete blocks. Later these pipes will be used to cool the concrete to open contraction joints for grouting.

Design Research—Wide Variety Discussed

The Design Research Session offered another well-rounded program to convention participants. Subjects of wide interest included effect of bar cutoff on bond and shear strength of beams, shear strength of continuous beams, torsional resistance of concrete, and behavior of continuous slabs prestressed in two directions. Henry M. Layne, consulting structural engineer, Los Angeles, presided as chairman.

Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams—PHIL M. FERGUSON, Department of Civil Engineering, University of Texas, Austin, and FARID N. MATLOOB, chief engineer, Municipality of Mosul, Iraq.

Cutting off reinforcing steel bars in tension zones is shown to lower the shear strength of beams substantially, whereas bending up of bars shows no such ill effect. It is indicated that both bond stress and diagonal tension are acting to bring about the reduced strengths.

Shear Strength of Two-Span Continuous Reinforced Concrete Beams—JOSE J. RODRIGUEZ, ALBERT C. BIANCHINI, and CLYDE E. KESLER, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, and IVAN M. VIEST, AASHO, Road Test, Ottawa, Ill.

The current design methods for proportioning reinforced concrete members take into account the external shear forces, cross-sectional dimensions, concrete strength, and web reinforcement. However, over 50 years ago Talbot demonstrated that the shear behavior of concrete members is affected also by the ratio of the span length to the effective depth of the member, and by the percentage of longitudinal reinforcement. More recent investigations have shown that these factors neglected in shear design have a major effect on the shear behavior of reinforced concrete members.

Most of the shear tests have been made on simple beams or on simply supported beams with overhangs. An application of the results of such tests to the design of continuous members involves the assumption that continuity has no effect on the shear strength. As most of the actual beams are continuous, it was considered necessary to determine to what extent this assumption is correct, or whether there are factors in continuity that have an effect on the shear behavior of statically indeterminate structures.

The primary objective of this investigation was to determine the effect of continuity on the shear strength of statically indeterminate structures. Two-span continuous beams were selected for their relative simplicity. The following variables were included: cutoff or extended longitudinal steel, percentage and spacing of web reinforcement, the type of loading, and the grade of longitudinal reinforcement.

The investigation was carried out in two parts. The first part was concerned with the shear strength of reinforced concrete beams with and without web reinforcement. Beams with web reinforcement were designed in accordance with the 1951 ACI Building Code. In this phase a total of 28 beams were tested. The second part of the investigation consisted of 24 beams, and was concerned with establishing the actual minimum amount of web reinforcement required to prevent shear failure.

Torsional Resistance of Reinforced Concrete—GEORGE C. ERNST, Civil Engineering Department, University of Nebraska, Lincoln. (Report presented by Mr. Jorgenson.)

Torsional stresses in reinforced concrete generally develop as a secondary effect of

flexure, such as in the spandrel beams of buildings. As a secondary effect, torsion is rarely a controlling factor in design, and therefore has received relatively little attention. Nevertheless, the increasing application of reinforced concrete to unique and unusual structural forms in which torsion may occur as a primary effect focuses attention on the need to develop a better understanding of torsional strength and stiffness in all situations.

Considering the problem of torsion as a whole, analytical design procedures are needed, supported by adequate experimental information for both conventional and prestressed reinforced concrete when subjected to static and dynamic loadings. Loading conditions must include combinations of torsion with flexure and direct stress over a broad enough range to cover all critical applications. The present extent of knowledge is limited to statically applied torques in combination with flexure.

Types of failures were discussed briefly and some of the work done to date, primarily the work of Cowan, was reviewed.

Behavior of a Continuous Concrete Slab Prestressed in Two Directions—A. C. SCORDELIS, T. Y. LIN, and R. ITAYA, Division of Civil Engineering, University of California, Berkeley.

Elastic behavior and ultimate strength of a continuous concrete slab prestressed in two directions were investigated. The slab was 15 ft square and 3 in. thick and was supported at nine points simulating column supports in a typical lift slab. Experimental deflections and moments due to prestressing and various loading conditions were compared with values obtained by the elastic plate theory and by approximate design procedures now in use. Observed ultimate strength was compared to that obtained by the crackline theory as applied to prestressed slabs.

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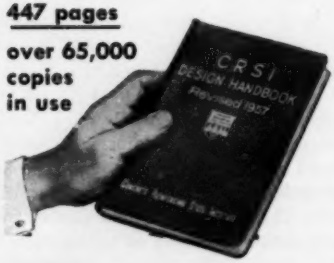

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Folded Plates and Decorative Panels Among Topics of Products and Precast Elements Session

Precast concrete was featured at the session at which Ernst Maag, principal structural engineer, Division of Architecture, State of California, was chairman. Precast, prestressed folded plates, and several kinds of decorative precast panels were among the subjects covered. Glazed face masonry units that combine structural strength and a good outward appearance were discussed. Also presented was a paper on linings for concrete pipe subjected to corrosion by sewage.

Precast and Prestressed Folded Plate Slabs—H. H. EDWARDS, president, Leap Concrete, Inc., Lakeland, Fla.

Prestressed precast folded plate concrete slabs offer tremendous potential for widespread use in buildings for roof, floor, and wall construction and in highway bridges.

A relatively low capital investment in prestressing facilities will produce a complete span range of folded plate products limited only by the ability to deliver and erect. The cost of folded plate structures promises to be competitive with all other building materials for long spans and heavy loads.

Prestressed folded plate offers the opportunity of bringing the advantages of concrete construction—strength, fire resistance, blast resistance, and low maintenance—into buildings of all price ranges, and in addition, makes it economically feasible to design for concrete construction in the 50- to 120-ft span range.

Plastic Forms for Architectural Concrete—J. A. HANSON, development engineer, Products and Applications Development Section, Portland Cement Association, Chicago.

Plastic forms are being used to obtain unusual and pleasing architectural effects in concrete. The architectural treatment may consist of both surface finish and pattern decoration. The concrete surface can be either glossy smooth or textured. The finish is achieved by thorough consolidation of the mix that would ordinarily be used in the product. Pattern decoration is achieved by vacuum forming the plastic sheet over wood, plaster, metal, glass or other pattern materials. Almost any intricate or bold pattern may be used, limited only by maintaining sufficient draft along pattern lines and maintaining the depth of depressions less than the width.

The extremely fine, smooth finish of plastic-formed concrete greatly improves the attractiveness of integral colors. Due to the higher reflectivity, smaller amounts of pigment are required to obtain a given color intensity.

The plastic is relatively expensive, but several reuses may be obtained. Laboratory tests of different types of plastics have indicated that at least ten reuses can be obtained with a reasonable amount of care in cleaning and handling the forms.

Exposed Aggregate Type of Decorative Panels—C. D. WAILES, JR., president, Wailes Precast Concrete Corp., Sun Valley, Calif.

Emphasis on color and texture in recent years has focused attention on the almost unlimited variation of both of these qualities as found in exposed aggregate concrete panels.

Although finely etched small aggregate is used for detail and tracery, the trend to bold, rugged use of relatively large aggregate exposed by removing the matrix $\frac{3}{8}$ in. to $\frac{3}{4}$ in. and more for flat fascia indicates the desire for "readable" textures at considerable distance and these provide interesting contrast to adjoining smooth or polished materials, metals, and glass.

Used as exterior forms against which concrete is cast or shot; applied as veneer, or installed as complete curtain walls these

panels may function structurally as well as decoratively.

Generally, natural aggregates are used but glass, ceramics, and other manufactured aggregate lend extra color, brillance, and interest when used.

Glazed Face Concrete Block—S. H. WESTBY, manager, Housing and Cement Products Bureau, Portland Cement Association, Chicago, Ill.

Glazed face concrete block provides a masonry unit that combines both structural strength and a glazed finish for use as a wainscoting in schools, public buildings, washrooms, kitchens, and laboratories. Such has been the rapid growth of this type of facing that it is challenging the market which once belonged exclusively to structural glazed tile.

There are several cold glaze cement finishes that are on the market at present which are a mixture of portland cement, graded silicas, mineral pigments, and chemical additives. These finishes can be applied on the units either before they leave the block plant or after a wall is completed on the job. A hard vitreous like surface is produced.

Some of the other types of glazed finishes are produced with resinous bases such as polyester combined with pigments or colored aggregates. Another type fuses ground glass and pigments to the face of the block producing a vitreous coating. Another bonds a terrazzo finish to the concrete block at the time of molding the block.

Glazed face concrete block are available in many markets and in a wide selection of color and shapes. Some of it can also be used for exterior purposes, but several are, at this time, limiting its use to only interior finishes until further tests and experience warrant otherwise. Its cost, of course, is one of its chief selling points. Some of these glazed face units cost as much as two to two and one-half times less than structural glazed tile. Others may come close to being equal in material cost on a per sq ft basis, however, the greater size of the concrete unit results in over-all lower cost.

Linings for Concrete Pipe and Structures Under Sewage Conditions—C. G. MUNGER, vice-president, Amercoat Corp., South Gate, Calif.

The subject of corrosion is one which has many ramifications, and most of these have been thoroughly discussed in recent years. The theory of electro-chemical corrosion has been established for metals and, in general, the majority of corrosion which takes place throughout the world follows this theory. Corrosion in sewers, however, does not necessarily conform or follow this theory, but is more a strict chemical reaction. Paper discussed concrete corrosion and methods used for protection against this corrosion.

Annual Research Session

There were 11 progress reports of current research presented at the annual research session. The session is sponsored by ACI Committee 115, Research as part of its work of reviewing and correlating research in concrete and reinforced concrete.

Reports of the following research were presented:

Dynamic Testing of Fixed-End Reinforced Concrete Beams in the Atomic Blast Simulator—Warren A. Shaw and J. R. Allgood, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

Effectiveness of Longitudinal Steel at Middepth as Diagonal Tension Reinforcement—James Chinn, University of Colorado, Boulder.

Continuity of Precast-Prestressed Bridges—Paul Kaar, Portland Cement Association, Chicago.

Torsional Resistance of Prestressed Concrete Members—Paul Zia, University of Florida, Gainesville.

Vibration Characteristics of Reinforced and Prestressed Concrete Beams—Gerald M. Smith, University of Nebraska, Lincoln.

RESEARCH COMPILATION

As part of its work, ACI Committee 115 each year assembles a list of research agencies doing work in the fields of concrete and reinforced concrete and various projects in progress. The 1959 compilation lists 91 organizations and over 600 projects in progress. The compilation is divided into three sections: educational institutions in the United States, noneducational institutions in the United States, and institutions outside the United States.

A limited number of compilations are available from ACI Headquarters for those who did not receive a copy at the Los Angeles convention.

The committee is anxious to increase the coverage of research activities here and abroad. Anyone having suggestions, or knowing of omissions from the compilation, should advise the secretary of the committee. Information will not be given publicity unless permitted by the correspondent. Address inquiries to

S. J. CHAMBERLIN
Secretary, ACI Committee 115
T. and A. M. Laboratory
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Ames, Iowa

Concrete Made with Portland Blast-Furnace Slag Cements—Paul Klieger Portland Cement Association, Chicago.

Cement Composition and Concrete Durability in Sea Water—Bryant Mather, Waterways Experiment Station, Vicksburg, Miss.

Determination of Water in Hydrated Cement by Use of Nuclear Magnetic Resonance —Raymond L. Blaine, National Bureau of Standards, Washington, D. C.

Drying Shrinkage Test Methods for Concrete Masonry—Rudolph C. Valore, Jr., Texas Industries, Inc., Dallas.

Detection of Incipient Fatigue in Mortar by Ultrasonics—John T. McCall, Michigan State University, East Lansing.

Accumulative Fatigue Damage in Plain Concrete—Hubert Hilsdorf, University of Illinois, Urbana.

Another important part of the work of Committee 115 is the compilation of research in progress by various agencies throughout the world. Each convention participant attending the research session was given a copy of this important document. A limited supply is available from ACI Headquarters for those desiring a copy.

ACI Annual Awards

At the Awards Luncheon on February 25, Douglas McHenry, retiring president, presented the ACI annual awards on behalf of the Board of Direction of the Institute. Election of honorary members was also announced.

Turner Medal

Douglas E. Parsons was awarded the Henry C. Turner Medal in "recognition of notable achievement in the execution and direction of research on concrete and in furtherance of preparation and acceptance of concrete construction standards."

Mr. Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D. C., has contributed to the field of concrete through many lines. His technical contributions have been in the fields of masonry construction and reinforced concrete. Author of some 30 papers, Mr. Parsons has encompassed the fields of reinforcement, compressive strength, shear strength, efflorescence, and water-tightness of masonry walls; Mesnager hinges; cement-water paints; and width and spacing of tensile cracks in reinforced concrete cylinders.

Active in ACI affairs since 1926, his record of achievement in behalf of the Institute presents an impressive picture. Culminating many years of activity with administrative and technical committees as well as the Board of Direction, he was vice-president in 1943 and 1944, and president of the Institute in 1945.

Henry L. Kennedy Award

The Henry L. Kennedy Award was given posthumously to Joseph Di Stasio, Sr., in

M. J. Greaves, on the right, received the Wason Medal for Research from President McHenry. His co-author, F. T. Mavis, was unable to be present to receive his award. Douglas E. Parsons, Turner Awardee, is in the foreground

recognition of his outstanding contributions to the advancement of the objectives of the American Concrete Institute. This award, established in 1958 in memory of the late Henry L. Kennedy, past president of ACI and chairman of the Building Committee, is given for outstanding technical or administrative service to the Institute.

Mr. Di Stasio as a partner in Di Stasio and Van Buren, New York engineering consultants, effectively promoted and widened the field of reinforced concrete construction through development of design methods, and devotion of time and effort in professional societies and code committees.

A graduate of Columbia University, Mr. Di Stasio was a recognized authority on reinforced concrete. He wrote numerous technical papers dealing with the subject and received many awards for his outstanding contributions. Much of his work was done with and through the medium of ACI.

Mr. Di Stasio joined the American Concrete Institute in 1920. He contributed many articles to JOURNAL pages and was an active participant in technical commitee activities, particularly ACI Committee 318, Standard Building Code. At the time of his death in September, 1958, Mr. Di Stasio was serving as chairman of Subcommittee 10 of Committee 318, and was a member of ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs.

Wason Medal for Research

F. T. Mavis and M. J. Greaves were awarded the Wason Medal for Research for their paper "Destructive Impulse Loading of Reinforced Concrete Beams" published in the September 1957 ACI JOURNAL. Founded in 1917 by the late Leonard C. Wason, this award is bestowed to an ACI member or members for the most noteworthy research or discovery reported in an Institute paper within a given publication year.

F. T. Mavis, dean of engineering, University of Maryland, was formerly head of the department of civil engineering, Carnegie Institute of Technology. Prior to 1944, he served as department head at the State University of Iowa and Pennsylvania State University. He was educated at the University of Illinois and Technische Hochschule, Karlsruhe, Germany, where he was one of the first Freeman Traveling Fellows of ASCE. Dean Mavis has written extensively in hydraulic engineering, structural analysis, and applied mechanics.

M. J. Greaves is presently engaged as development engineer for Arthur G. McKee and Co., Cleveland. He formerly was professor of civil engineering at Utah State Agricultural College. As a major in the Panama Canal Department during World War II, Mr. Greaves supervised military construction projects. His experience has included many unusual problems in reinforced concrete design and stress analysis. Mr. Greaves was graduated from Cornell University in 1940, and received his PhD in 1956 from Carnegie Institute of Technology.

Wason Medal for Most Meritorious Paper

John F. Brotchie was awarded the Wason Medal for Most Meritorious Paper for his paper "General Method for Analysis of Flat Slabs and Plates" published in the July 1957 ACI JOURNAL. This award, established in 1917 by Mr. Wason, is presented for the most meritorious paper by a member or members of the Institute published in the latest volume of ACI Proceedings.

President Douglas McHenry presents awards to John F. Brotchie (left) and Jack E. Rosenlund. Mr. Brotchie received the Wason Medal for the most meritorous paper and Mr. Rosenlund received the Construction Practice Award

Mr. Brotchie, on graduation from Melbourne University in 1950 with a degree in civil engineering, accepted a position with the Commonwealth Department of Works, Head Office, Melbourne. He spent his first year designing roads and airfields, then turned his attention to the design of structures in steel, reinforced concrete, and prestressed concrete. Mr. Brotchie is now on leave from the Department of Works to attend the University of California, Berkeley, as a graduate research engineer with the Institute of Engineering Research.

Construction Practice Award

Jack E. Rosenlund received the Construction Practice Award for his paper "Construction of the Dallas Memorial Auditorium" published in the October 1957 ACI JOURNAL. This award was founded in 1944 to help enrich the literature of that field of endeavor and to honor the construction man—the man on the job—for his resourcefulness in translating design into a completed structure.

Mr. Rosenlund is chief structural engineer for George L. Dahl Architects and Engineers of Dallas. He has been with the organization since 1949. He graduated from Southern Methodist University in 1944 and received his MS in civil engineering from the University of Minnesota in 1948.

Honorary Members named

Honorary membership in the American Concrete Institute was conferred on three eminent men in the field of concrete: the late Hardy Cross, Yale University, New Haven, Conn.; H. J. Gilkey, Iowa State College, Ames; and Eduardo Torroja, Madrid, Spain.

Hardy Cross, professor emeritus of civil engineering, Yale University, was probably best known for his contribution to structural engineering of two methods of analyzing indeterminate structures. These methods, generally known as the "column analogy and the moment distribution method," received world wide attention.

H. J. Gilkey, professor and formerly head of the Department of Theoretical and Applied Mechanics, Iowa State College, has long been eminent in the field of research in engineering materials, especially concrete. He has written several textbooks and manuals, and numerous technical papers and discussions dealing with concrete, engineering materials, and testing methods, many of which have appeared in the ACI JOURNAL.

Eduardo Torroja, director of Instituto Technico de la Construccion y del Cemento, Madrid, Spain, has published over 40 papers and books, and has designed about a thousand projects. The Technical Institute, which he founded, has been referred to as one of the

world's three outstanding model testing laboratories. The results of tests at the Institute have formed the basis for most of Professor Torroja's own complicated designs.

Student competition

The Southern California Chapter of ACI and the local convention exhibits committee sponsored a student competition awarding $400 to the winners at the annual awards luncheon of the Institute on February 25.

First prize of $200 was awarded Yin Li, fifth year student at the University of Southern California, from Peking, China, for his renderings of a shopping center. Second prize of $100 went to a fourth year student at the University of Southern California, Norman Lacayo, from Managa, Nicaragua, for his church design.

A local Los Angeles miss, Ena Dubnoff, took honorable mention and a $50 award for her renderings of a museum. Miss Dubnoff is a fourth year student at USC. A similar award was made to Gene Zellmer also a fourth year student at the university from Fresno, Calif., for his renderings of a zoo building.

The competition was open to all students in southern California colleges regularly enrolled in engineering or architecture.

The winners were selected from some 25 entries by a panel of four judges prominent in the building industry in southern California. Judges included: Henry Wright, AIA, partner in the architectural firm of Kistner, Wright and Wright; Henry M. Layne, consulting structural engineer; Ernst Maag, principal structural engineer of the California Division of Architecture; and Emil Wohl of Wohl-Calhoun Construction Co. Entries were in the form of scale models, architectural renderings, perspective drawings, and pictorial presentations.

Ferguson 1959 ACI President

Phil M. Ferguson, professor of civil engineering, University of Texas, Austin, was elected for a 1-year term as president of the Institute, succeeding Douglas McHenry, director of development, Portland Cement Association, Skokie, Ill.

Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California Department of Water Resources, Sacramento, was elected to a 2-year term as vice-president of the Institute. Joe W. Kelly, professor and vice-chairman of the department of civil engineering, University of California, Berkeley, elected to a 2-year term as vice-president in 1958 continues in that position.

Four new directors were elected to 3-year terms. The new members of the Board of Direction are: Roger H. Corbetta, president, Corbetta Construction Co., New York; George C. Ernst, professor of civil engineering and director of the Engineering Experiment Station, University of Nebraska, Lincoln; E. A. Finney, director of the research laboratory, Michigan State Highway Department, Lansing; and Walter J. McCoy, director of research, Lehigh Portland Cement Co., Coplay, Pa.

President Ferguson

Professor Ferguson has just completed his second year as ACI vice-president. His achievements reveal his participation in Institute affairs since becoming a member in 1930. He has served on the Board of Direction, the Technical Activities Committee, and numerous technical and administrative committees. He is presently a member of ACI Committee 115, Research; Committee 208, Bond Stress; and Committee 318, Standard Building Code. He is ACI representative on

the American Welding Society subcommittee on welding of bars, metal inserts, and connections in reinforced concrete construction.

Professor Ferguson, who holds two degrees from the University of Texas, recently relinquished the chairmanship of civil engineering department there, so that he might devote more time to teaching and research. Long recognized as an authority on structural design, Professor Ferguson has been consultant on a number of projects.

Vice-President Tuthill

Lewis H. Tuthill has participated in nearly every phase of Institute activity since he became a member in 1926. He has been a member of the Board of Direction, the Technical Activities Committee and numerous ACI technical committees. Mr. Tuthill has prepared or collaborated on at least 14 papers for the ACI JOURNAL. Two of his contributions brought him special Institute recognition, the Construction Practice Award (1947) and the Wason Medal (1956).

Mr. Tuthill is chairman of ACI Committee 611, Inspection of Concrete, and Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete. He is a member of five other ACI technical committees.

A graduate of Oregon State College, Mr. Tuthill has been engaged in design and construction work on irrigation, water supply projects, and the building of dams throughout his career. His special interest has been the technique and control of concrete.

New Directors

Roger H. Corbetta, has been an ACI member since 1941. He has been identified with precast concrete structures and prestressing methods since 1920 and has pioneered in the field of thin shell concrete construction.

One of the founders of the Concrete Industry Board of New York City, Mr. Corbetta served as chairman of the organization for 5 years and is now the Institute's representative on the board. He is also a member of the ACI Committee 324, Precast Reinforced Concrete, Thin Sections. He is a director of the National Concrete Contractors Association, and a member of the board of governors of the Building Congress of New York City.

Retiring President Douglas McHenry (right) turned over his office and unfinished projects to incoming President Phil M. Ferguson with a "symbol" of his new office—the eight ball

George C. Ernst, has been an ACI member since 1935, actively participating in administrative and technical committee work.

Professor Ernst has contributed a number of technical papers which have appeared in the ACI JOURNAL. He is currently a member of ACI Committee 115, Research, and will serve as chairman of the newly formed Committee 338, Torsion.

E. A. Finney, is presently a member of the Technical Activities Committee; chairman of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports; a member of Committee 116, Nomenclature; and ACI representative to the Highway Research Board. He has been an ACI member since 1944.

Mr. Finney has been especially active in research associated with air-entrained concrete, concrete pavement design and performance, and concrete materials.

Walter J. McCoy has been active in ACI since joining in 1946. He completed a 3-year term as a member of the Board of Direction in 1958 and is currently chairman of the Technical Activities Committee. Mr. McCoy is a member of ACI Committee 115, Research; 116, Nomenclature; 201, Durability of Concrete in Service; 216, Fireproofing or Fire Protection of Structures; and 331, Structures of Concrete Masonry Units. He is also ACI representative to ASA Sectional Committees A-41 and A-42, on masonry and plastering, respectively.

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Nominating Committee

The newly elected nominating committee consists of T. B. Kennedy, chief, Concrete Division, Waterworks Experiment Station, Corps of Engineers, Jackson, Miss.; T. C. Powers, research counselor, Portland Cement Association, Chicago; C. P. Siess, professor, Civil Engineering Division, University of Illinois, Urbana; Anton Tedesko, vice-president, Roberts and Schaefer, New York City; and A. R. Anderson, consulting engineer, Anderson, Birkeland and Anderson, Tacoma, Wash.

Also serving on the committee will be the three most recent past presidents of the Institute, Frank Kerekes, dean of the faculty, Michigan College of Mines and Technology, Houghton; Walter H. Price, chief, Engineering Laboratories, U. S. Bureau of Reclamation, Denver; and Douglas McHenry, director of development, Portland Cement Association, Skokie, Ill.

Local Committee Sponsors Special Events

Augmenting an outstanding technical program, the Los Angeles Convention Committee arranged an impressive schedule of entertainment events with ladies given prime consideration in the entertainment planning.

All conventioneers were invited to a gigantic get-acquainted social gathering on Wednesday evening with a "Pacific island" atmosphere. Both delegates and wives visited the world-famous Disneyland on Friday, February 27, following the conclusion of the official convention program.

Special features for the ladies included a get-acquainted bruncheon on Tuesday morning, February 24, and a fashion luncheon at the Beverly Hilton Hotel, Beverly Hills, with fashion show staged by Walter Plunkett, Academy Award-winning designer for Metro-Goldwyn-Mayer Studios, followed by a short tour of the Beverly Hills area.

Los Angeles convention committee

Samuel Hobbs, secretary-treasurer of the Southern California Chapter of ACI, served as general chairman of the local planning committee for the convention with William F. Norton, Ceco Steel Products, acting as treasurer of the committee. Assisting on the local general committee were: Ross Adams, Portland Cement Association; Henry M. Layne, consulting structural engineer, Ernst Maag, California Division of Architecture; John McNerney, Portland Cement Association; Lewis K. Osborn, Kistner, Wright and Wright; O. C. Struthers, Raymond Concrete Pile Co.; C. Taylor Test, Riverside Cement Co.; Glenn C. Thomas, Thomas Concrete Accessory Co.; Byron P. Weintz, Consolidated Rock Products Co.

Entertainment and Hospitality Committee

C. Taylor Test, Riverside Cement Co., served as committee chairman with R. H. Cooley, Portland Cement Association, assisting as vice-chairman. Others working on the committee were: James C. Bugg, California Portland Cement Co.; W. Robert Blair, Southwestern Portland Cement Co.; Richard G. Hames, Monolith Portland Cement Co.; Phili. A. Harrigan, Permanente Cement Co.; Marvin J. Kudroff of Daniel, Mann, Johnson and Mendenhall; Charles Pankow, Peter Kiewit Co.; and John C. Wise, Disneyland.

Exhibit Committee

Glenn C. Thomas, Thomas Concrete Accessory Co., headed the committee assisted by Robert E. Tobin, Portland Cement Association, in charge of student competition; C. F. Brown, Master Builders Co.; John L. Goetz, Southwestern Portland Cement Co.; George C. Guibert, Super Concrete Emulsions; and John G. Kasnetsis, Hunt Process Co.

Reception Committee

Henry M. Layne, consulting structural engineer, was chairman assisted by Ernst Maag, California Division of Architecture; Harald Omsted, Los Angeles City Board of Education; Joseph Sheffet, consulting structural engineer; C. D. Wailes, Jr., Wailes Precast Concrete Corp.; George E. Warren, Southwestern Portland Cement Co.; and William T. Wright, Kistner, Wright and Wright.

Ladies Events

Mrs. C. Taylor Test headed up the ladies contingent with Mrs. R. H. Cooley assisting as vice-chairman.

Mrs. Charles J. Pankow acted as chief hostess at the ladies bruncheon with Mesdames Ross Adams, John J. Driskell, Samuel Hobbs, Ernst Maag, Harald Omsted, Joseph Sheffett, Byron P. Weintz, and C. D. Wailes, Jr., acting as hostesses.

Mrs. Marvin J. Kudroff acted as chief hostess at the fashion luncheon assisted by Mesdames W. Robert Blair, W. C. Bryant, Felix Kulka, Henry M. Layne, John M. McNerney, J. L. Peterson, and Robert E. Tobin.

Construction Information

John M. McNerney headed up the construction information committee assisted by George E. Battey, Jr., Portland Cement Association; J. L. Peterson, J. L. Peterson, Inc., and Byron P. Weintz, Consolidated Rock Products Co.

Hotel Arrangements

Lewis K. Osborn of Kistner, Wright and Wright, assumed the chairmanship of this committee assisted by W. C. Bryant, Portland Cement Association.

Membership

O. C. Struthers, Raymond Concrete Pile Co., chairman, was assisted by Charles M. Dabney, Admixtures, Inc. and Richard C. Hodson, L. M. Scofield Co.

Convention Exhibitors

One of the highlights of the convention was the product displays. Thirty-eight organizations were represented and attendance to the exhibits was reported to be excellent throughout the convention. Company representatives were present at each of the booths and catalogs, specifications, and technical literature was available. Those exhibiting at the convention included:

Admixtures, Inc., Pasadena, Calif.—Western states distributors of "Maracon," a water-reducing admixture for concrete produced by Marathon Corp. of Wisconsin. Photos depicting the use of Maracon on construction projects were displayed along with fact sheets and technical data.

Autolene Lubricants Co., Denver, Colo.—A display featuring "Protex" air-entraining admixture for concrete. Photos on display showed construction where Protex was used. Technical literature available included placement of air-entrained concrete and data on "Probond" concrete bonding agent. Demonstrations of a Protex air meter were handled by company representatives.

Concut Sales, Inc., El Monte, Calif.—This display featured various concrete saw blades with a pictorial emphasis on the "Concut Jointmaster," a hydraulic drive sawing machine available for large airbase and highway sawing jobs. Technical data on a "Concut Bump Cutter" and the firm's complete line of concrete sawing equipment were available.

Dewey and Almy Chemical Division, W. R. Grace Co., Cambridge, Mass.—A backdrop displaying the use of "Dara Weld" bonding agent. The exhibit itself was basically established for technical narration and supplying technical literature.

Ducore, Inc., Los Angeles, Calif.—"Concore" test core cutting equipment was displayed with supporting technical information as to the various uses for test core equipment. Ducore, Inc., is a core testing service organization.

A walk around the exhibit hall

Edoco Technical Products, Inc., Long Beach, Calif.—Prominence was given to a backdrop listing uses and facts about various Edoco concrete construction materials and concrete specialties. Main feature was the "E-Z-Seal," polyvinal chloride waterstop for expansion and construction joints. Technical literature included a catalog of Edoco products.

Encyclopaedia Brittanica, Los Angeles, Calif.—A display built around a special convention offer for an entire set of the Encyclopaedia Brittanica.

Food Machinery Co., Riverside, Calif.—"Form-Crete" steel forms were on display with a brochure explaining the various uses of these forms. A special feature of the display was a "Lin single T" element for exterior and interior wall construction.

Furane Plastics, Inc., Los Angeles, Calif.—The display featured technical bulletins on "Epocast" coatings as sealants, adhesives, and pastes for the concrete industry. The lighted backdrop emphasized their epoxy systems, with photos displaying these systems in use.

Gar-Bro Manufacturing Co., Los Angeles, Calif.—A miniature concrete bucket and a receiving hopper were the main objects of this display. Photos displaying concrete handling equipment in use served as a backdrop for the physical display. Technical literature on concrete handling equipment was also distributed.

General Concrete Products, Inc., Van Nuys, Calif.—Various concrete masonry screens were displayed in different colors, showing various uses for commercial and residential construction. A brochure showing 15 different block designs was distributed.

Great Books of the Western World, Los Angeles, Calif.—An exhibit and special offer of 54 volumes spanning western thought from Homer to Freud. The collection contains whole works and is published by the Encyclopaedia Britannica.

Gunite Contractors Association, Los Angeles, Calif.—A display of literature on the uses of "Gunite" and the purpose of the association.

Hunt Process Co., Inc., Los Angeles, Calif.—A pictorial display of the use of the various Hunt processes for curing concrete. Photographs showed construction projects using their curing compounds, and literature was available describing the various compounds available.

International Processing Corp., Los Angeles, Calif.—A photographic and physical display of "Ductube" and "Contiduct," rubber tubing used for placing concrete pipe and duct formations by inflating and deflating after the concrete is in place. A "Myton" fully automatic ram designed for single-strand tensioning for pretensioning or post-tensioning of prestressed members was illustrated.

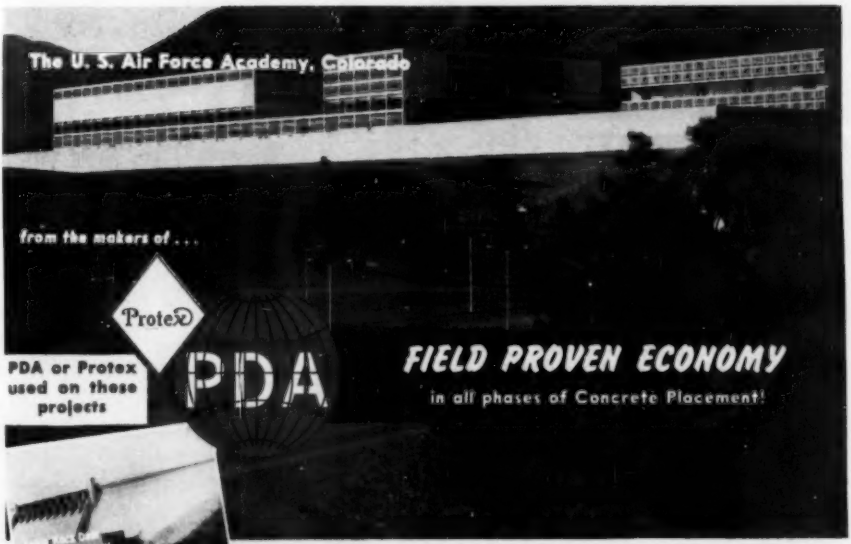
The Master Builders Co., Division of American-Marietta Co., Cleveland, Ohio—Literature and pictorial display of their products with emphasis given to "Pozzolith" admixture. Intricate triangular moving blocks with construction photos created the center of attraction and displayed actual uses of Pozzolith.

Neptune Manufacturing Co., Los Angeles, Calif.—Neptune screed supports, reinforcing wire mesh aluminum chairs, curb stake brackets, and tilt-up accessories were displayed. Technical data applicable to the various methods of use were available.

Portland Cement Association, Los Angeles, Calif.—Lighted backdrop behind full-color photos of concrete in use was the main attention getter. Various masonry units and plastic forms for decorative concrete work were also displayed.

The Prescon Corp., Gardena, Calif.—Full-color projected slides were the center of attention at this display, illustrating the "Prescon" system of prestressing. Other black and white photos on either side of the projector depicted projects around the country that used this system.

Remington Arms Co., Inc., Bridgeport, Conn.—A physical display of Remington stud drivers, concrete vibrators, and flexible shaft machines for grinding, sanding, polishing, wire brushing, and rotary filing. Demonstrations of the equipment were conducted on request and technical data were available.



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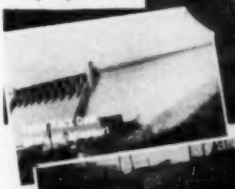

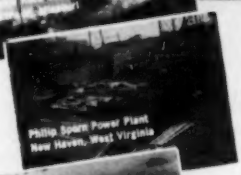

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Rocklite Products, Los Angeles, Calif.—A photo-mural of the manufacture of expanded shale aggregates was used as a backdrop for a display of various kinds of expanded shale aggregate and a number of different methods of using this material for special masonry units. A brochure describing these various masonry units was also available.

Sika Chemical Corp., Passaic, N. J.—Sika "Plastiment" and "Sikacrete" were featured with pictorial support of actual use of these products. An interesting physical display was a section of concrete where "Rugasol" retardant concrete coating had been used to expose aggregate. Technical data were distributed.

Sonoco Products Co., Construction Products Division, Hartsville, S. C.—"Sonotube" fiber forms were displayed with an explanation for their use in construction. A backdrop of Sonotube fiber forms and "Sonovoid" fiber tubes was displayed with facts and a pictorial record of construction projects utilizing these products.

Stow Manufacturing Co., Binghampton, N. Y.—Stow vibrators, grinders, trowels, and screeds were displayed along with technical data covering all Stow products.

Superior Concrete Accessories, Inc., San Leandro, Calif.—Various form ties and clamps were displayed with explanations for their use. Information on other Superior products was distributed.

Surface Engineering Co., Inc., Wichita, Kan.—This display consisted of a model of the new ACI Headquarters Building and the application of "Seco" plastic protective coating on its roof. Technical data were available with a display of sections of concrete showing actual application of the plastic coating.

Symons Manufacturing Co., Chicago, Ill.—Symons shores and column clamps were displayed along with technical data covering the use of Symons "steel-ply" forms for various applications.

Thomas Concrete Accessory Co., Los Angeles, Calif.—A display of Thomas "vinlhesive," an adhesive for patching concrete; concrete curing compounds; metal form and

screed stakes; and form protection solution. Explanations of their uses were made and technical literature was also available.

E. A. Thompson Co., Inc., Los Angeles, Calif.—"Thompson's Water Seal," a waterproofing surface sealing for tile, wood, brick, metal, plaster, and concrete was displayed. Technical data were distributed.

True Gun-All Equipment Co., Tulsa, Okla.—Prominence to the display was given by the True Gun-All pneumatic concrete machine model F-2. Photos of the use of pneumatic concrete in residential, municipal, industrial, and rural applications served as a backdrop for the machine. Technical data were available on the uses of the different machines.

U. S. Plywood Corp., Los Angeles, Calif.—The most prominent element of this display was the different panels of "Weldwood" plywood for use as reusable concrete forms. Each panel gives its own particular texture that can be used for decorative purposes.

Viber Co., Burbank, Calif.—A display of different models of the "Univiber" motor-in-head internal vibrator. Technical literature on vibration of concrete and the use of the company's vibrators was distributed.

Water Seals, Inc., Chicago, Ill.—The various waterstops of the company were displayed along with a technical data sheet on the uses of these waterstops. A booklet describing the "Labyrinth Waterstop" was also distributed.

Fibrecrete Corp., Long Beach, Calif. and **Professional Publishing Co.,** Pasadena, Calif.—A cooperative booth displaying photos of marine floating concrete docks with technical drawings of their construction for the Fibrecrete Corp. The Professional Publishing Co. displayed technical literature on various phases of concrete and concrete construction.

Model Contest Display—Sponsored by the California Portland Cement Co., Monolith Portland Cement Co., Permanente Cement Co., Riverside Cement Co., and the Southwest Portland Cement Co. A display of renderings and models submitted by architectural and engineering students from 15 southern California colleges, using portland cement concrete in their construction. Over 25 entries were on display with two winners and two honorable mentions being picked to receive $400 in cash awards.

California State Freeway Model—Sponsored by Guy F. Atkinson Co., J. E. Haddock, Ltd., Raymond Concrete Pile Co., Webb and White, and Peter Kiewit Sons' Co. A model of the current and proposed California freeways in and around Los Angeles.

U. S. Bureau of Reclamation—Sponsored by Southern California Ready-Mixed Cement Association, and Southern California Rock Products Association. A map of western United States showing the location of dams that had been or are being built by the Bureau of Reclamation for water control. A booklet, "Dams for Reclamation" was also distributed.

LOOKING AHEAD

May 4-8, 1959—Cleveland Meeting, American Society of Civil Engineers, Hotel Cleveland, Cleveland

May 13-15, 1959—"Deutsche Betontag 1959," German Concrete Association, Congress Hall of the German Museum, Munich, Germany

May 18-19, 1959—39th Annual Meeting, The Society of American Military Engineers, Washington, D. C.

May 25-30, 1959—Annual Meeting, Concrete Reinforcing Steel Institute, The Greenbier, White Sulphur Springs, West Va.

June 21-26, 1959—Annual Meeting American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

Nov. 3-5, 1959—Regional Meeting, American Concrete Institute, Hotel Continental Hilton, Mexico City

Voids Reduce Weight and Maintain Rigidity in Warped Slab

Garage ramp of Pacific Telephone & Telegraph office, Oakland, Calif. Ira Beals, architect; Pregnoff and Mather, engineers; R. F. Royden, contractor.

Form voids in concrete slabs with low-cost SONOCO SONOVOID FIBRE TUBES

Weight became a problem in the design of this concrete garage ramp. To provide required structural rigidity, the warped slab had to be extra thick in depth . . . extra weight was reduced by planning a voided slab system.

Using low-cost 12" O.D. SONOVOID Fibre Tubes to form voids, the dead weight of low-working concrete was eliminated, with slab strength and rigidity maintained.

SONOVOID Fibre Tubes are specifically designed for use in bridge decks, floor and roof slabs, and in concrete piles. For precast, pre-stressed units or members cast in place. The voided slab system means savings in concrete and reinforcing steel.

Sonoco Fibre Tubes are available in sizes from 2.25" to 36.9" O.D. Order in specified lengths or standard 18' lengths. Can be sawed to your requirements on the job. End closures available.

Write for complete technical information and prices.
See our catalog in Sweet's

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- BRANTFORD, ONT.
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Positions and Projects

ACI Northern California Chapter conducts first annual meeting

The first annual meeting of the ACI Northern California Chapter was held in San Francisco, February 17.

Lewis H. Tuthill, California Division of Water Resources, Sacramento, featured speaker of the evening, addressed an audience of approximately 70 members and guests on the subject of "The Need for Concrete Inspection." Distinguished guests attending the meeting were Douglas McHenry, Portland Cement Association, Chicago, and Frank Kerekes, Michigan College of Mines and Technology, Houghton, Mich., 1958 and 1956 Institute presidents, respectively.

Results of the Chapter's recent election named S. D. Burks, Dewey and Almy Chemical Co. as president and H. E. Thomas, Pacific Gas and Electric Co., vice-president. Wilbur Moulton, Pacific Cement and Aggregates, Inc., was appointed to serve as secretary-treasurer.

Six directors were elected: J. A. Dobrowolski, Portland Cement Association; Ben C. Gerwick, Jr., Ben C. Gerwick Co.; E. L. Howard, Pacific Cement and Aggregates, Inc.; J. E. Jellick, Portland Cement Information Bureau; M. R. Neumann, Testing Engineers, Inc.; and Carl M. Rollins, Basalt Rock Co.

The 1959 Nominations Committee is headed by J. W. Kelly, University of California, as chairman, with Orville Jack, Permanente Cement Co.; F. R. Killinger, Hales Testing Laboratories; Alexander Klein, University of California; and Eric C. Moorehead of Smith and Moorehead, serving as committee members.

Southern California Chapter celebrates first year

The first annual meeting of the ACI Southern California Chapter since completion of organization and incorporation was held at Rodger Young Auditorium in Los Angeles on March 9. It was a joint dinner meeting attended by ACI Chapter members and members of the Southern California District Council of ASTM with about 115 attending.

Retiring president Ernst Maag, Division of Architecture, State of California, presided at the business meeting, announcing returns of the Chapter's recent election of officers for 1959.

J. L. Peterson, of J. L. Peterson, Inc., was elected president and Dewain R. Butler, Integrated Construction and Engineers, Inc., was elected vice-president.

Myron B. Niesley, California Testing Laboratories, Inc. and Glenn C. Thomas, Thomas Concrete Accessory Co., were elected directors for a 1-year period; C. C. Dubbs, Carbon Dubbs Co. and E. M. Markell, Airox Co., were elected directors for 2-year terms; John M. McNerney, Portland Cement Association and Lewis K. Osborn, of Kistner, Wright and Wright, were elected directors for 3-years.

Chairman of the committee on nominations was E. A. Paterson, Rocklite Products, with Harold P. King, structural engineer; Floyd E. Weaver, structural engineer; Charles J. Pankow, Peter Kiewit Co.; D. F. Roberts, Smith-Emery Co.; and Ernst Maag as past president serving as committee members.

Byron P. Weintz, Consolidated Rock Products, served as Program Chairman for the meeting. Mr. Weintz is chairman of the Southern California District Council of ASTM. He was a member of the original organizing committee during formation of the Southern California Chapter, and has just completed a term as director and member of the executive committee.

Among prominent guests introduced by Mr. Weintz were ASTM President K. B. Woods and ASTM Secretary Robert Painter. Other guests included: Col. John Oswalt of the Corps of Engineers; Edward Telford of the State Division of Highways; George E. Warren, Archie L. McCall, D. C. Honey, H. D. McBride and C. W. McKinley, executives of cement companies in southern California.

K. B. Woods, School of Civil Engineering, Purdue University, speaker of the evening, spoke on Polar construction, describing the problems of arctic construction projects and

the ways in which these problems have been met. He enlightened the audience concerning the nature of permafrost, its engineering properties, and its effects on design and construction techniques and methods.

Samuel Hobbs, chapter secretary-treasurer, reported total membership at the end of the year 1958 of 173: 4 Contributing; 17 Corporation; and 152 Individual members. Since Jan. 1, 1959, 3 Corporation and 11 Individual members have been added. A total of 58 ACI members have been added to the Institute roster through the establishment of the Southern California Chapter.

Mehoke joins JOURNAL staff

Richard A. Mehoke, a 1958 graduate of Michigan State University, East Lansing, has joined the ACI staff as assistant editor of the JOURNAL.

Mr. Mehoke majored in civil engineering at MSU. On graduating, he worked for the U. S. Lake Survey, Detroit, as assistant chief of party with the off-shore party. He has worked in automobile plants in Detroit and as a detail draftsman in the engineering department of Huck Mfg. Co., Detroit.

During 1952 and 1953 Mr. Mehoke served with an army engineer aviation battalion.

Iowa Concrete Conference

The 10th Annual Better Concrete Conference, sponsored by Engineering Extension and presented by the Department of Theoretical and Applied Mechanics, Iowa State College, featured a diversified program, March 12-13, on the Iowa State campus in Ames. About 100 persons registered for the conference.

The program included progress reports on the Iowa interstate highway program and on the AASHO road test, and a paper on silicones for highway construction. Prestressed concrete was covered in three talks: prestressed concrete on the Pacific Coast; status of prestress research at Iowa State College; and applications of prestressed concrete and thin shells in the Midwest. Other speakers presented reports on bar spacing in concrete beams, prepacked concrete, practical aspects of placing and maintenance in residential and commercial construction, control of concrete quality, crack control in plastic and hardened concrete, and a review of current laboratory studies being conducted by the National Ready Mixed Concrete Association. Job problems and solutions were the topic of another speaker, and reports were given on concrete technology in Russia and buildings at the Brussels World Fair. The recent ACI Los Angeles convention was summarized as was the National Ready Mixed Concrete Association convention in New Orleans.

Winners of the C. W. Shirey student competition were also announced. The awards were presented to students for their presentation and solution to the general problem of a shopping center with a roof parking deck utilizing prestressed concrete in the structure.

The annual Better Concrete Conferences are designed to contribute to improved concrete practices through semiformal programs, floor discussion, and the mingling of practitioner and theorist. Cooperating organizations included: ACI, Portland Cement Association, Iowa Highway Commission, Iowa Ready-Mixed Concrete Association, Iowa Engineering Society, American Institute of Architects (Iowa), American Society of Civil Engineers (Iowa), Master Builders of Iowa, and Associated General Contractors of Iowa.

J. C. Witt affiliates with Florida architectural firm

J. C. Witt, Chicago consulting engineer specializing in cement and concrete, has affiliated with Maurice H. Connell and Associates, Miami architectural-engineering firm.

Dr. Witt, long time ACI member (1924), is author of *Portland Cement Technology* and holds a series of patents dealing with the manufacture and use of portland cement.

Maurice H. Connell, president of the Miami firm announced that Dr. Witt would serve as an associate consulting engineer with his company but that he would continue to operate his Chicago office.

Ford represents Carolina Solite

A. Cabell Ford, Jr. has been appointed sales representative for the new Raleigh Office of Carolina Solite Corp., a wholly-owned subsidiary of the Southern Lightweight Aggregate Corp., Richmond, Va. This announcement was made recently by A. Cabell Ford, director of sales for Southern Lightweight Aggregate Corp.

AASHO Road Test fleet covers first million miles of pavement

AASHO Road Test officials report that a fleet of 70 trucks has accumulated a million miles of operation on five closed loops of test pavements, carefully constructed in sections of various thicknesses. Several more millions of miles will be chalked up before the test ends in the Fall of 1960. The nationally-supported test, sponsored by the American Association of State Highway Officials (AASHO), has been under way at Ottawa, Ill., since November.

The test pavements in the traffic loops are built in 716 separate sections with varying structural designs, different combinations of thicknesses of subbase, base, and pavement material. Half of each test loop is paved with portland cement concrete and half with asphaltic concrete.

Trucks, which range from pick-ups to huge tractors and semi-trailers, operate in two driving shifts per day, covering about 18 hr, six days a week.

In full operation, there are 60 vehicles running in ten lanes of five traffic loops. All vehicles in any lane have identical axle loads. The loads range from 2000 lb on the single axles of pick-ups to 48,000 lb on the tandem axles of big combination units.

Two of the traffic loops also contain test bridges. There are sixteen 50-foot spans; eight with steel beams, four with reinforced concrete beams, and four with prestressed concrete beams.

As traffic rolls on the five loops, a sixth one with 120 additional test sections, stands relatively idle. It is being used for special studies under static loads and to observe the effects of weather alone on highway pavements.

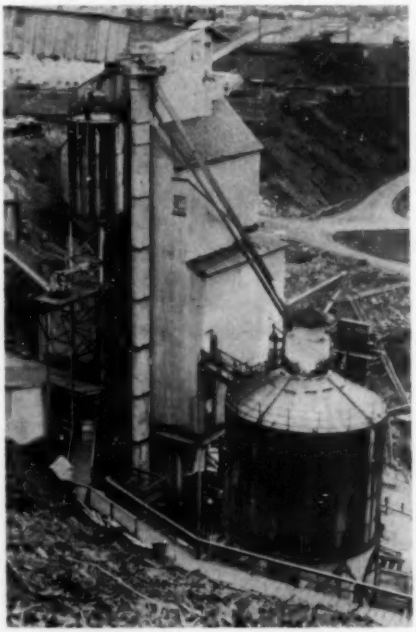
Some 7000 gages are installed on, in, and under the pavements to allow measurement of strains, deflections, pressures, temperatures, and frost depths. The output of these transducers is recorded automatically on instruments installed in movable vans.

Final findings of the project will be published in 1961 by the Highway Research Board. Walter B. McKendrick, Jr., former chief engineer of the Delaware Highway Department, heads the Board's staff on the project.

Multiple batching

EIGHT SEPARATE WEIGH BATCHERS for individual aggregates, cement, and water are in operation at this batching plant which supplies concrete for construction of Baker Dam Hydroelectric Project, owned by Puget Sound Power and Light Co. in the state of Washington. About 640,000 cu yd of concrete will be produced in the 2½-year project at an average of 150 cu yd per hr.

The plant has three 4 cu yd tilting mixers and an 8 cu yd wet mix holding hopper. Concrete is carried on cableways in 8 cu yd buckets to the placing site. Overhead aggregate storage totals 500 tons in six compartments; 500 bbl cement is stored overhead and another 5000 bbl is in ground storage. Batching and recording of materials is fully automatic, with mix selector for six different batch formulas. The Noble Co., Oakland, Calif., manufactured the batching plant.



Nordby represents structural engineers at research conclave

Gene M. Nordby, head of the department of civil engineering at the University of Arizona, Tucson, represented the field of structural engineering at a national conference on architectural research held at the University of Michigan, Ann Arbor, March 8-10.

Sponsored jointly by the American Institute of Architects and National Science Foundation, the conference included in its membership 40 leaders in the field of engineering, architecture, sociology, psychology, geography, medicine and public health, city planning, and civil and structural engineering.

Dr. Nordby, a member of the steering committee of the meeting, said that the theme of the conference was the place of research in the architect's use of space, "the optimum arrangement and conditioning of space for all kinds of human activities."

Dickinson named vice-president of Calcium Chloride Institute

George H. Kimber, president, announces the election of William E. Dickinson to the office of vice-president of the Calcium Chloride Institute.

Mr. Dickinson, who has been associated with the institute for 13 years, has been chief engineer since 1955 and will continue his responsibilities in that post.

University of Wisconsin engineering institutes

The University of Wisconsin Extension Division has scheduled a series of 2- to 5-day engineering institutes from February 5 to June 5 to present current information to technical personnel from industry. Material is presented by informal talks, panel discussions, film, slides, and demonstrations. The number in attendance at any one session is limited to the group size most appropriate to the subject matter and method of presentation, varying from 15 to 90 persons. Annual attendance at these institutes in previous years has been over 1700. Detailed program available from Engineering Institutes, 3030 Stadium, University Extension Division, University of Wisconsin, Madison 6, Wis.

Abell-Howe consolidates operations at Forest Park

Complete consolidation of all engineering, designing, construction, sales, and office facilities was affected recently by Abell-Howe Co. according to M. O. Haeger, president. New executive building in Forest Park, Ill., next to the steel fabricating plant of Abell-Howe, now becomes headquarters for all company operations and services.

PCA literature wins three honor awards

Literature and advertising produced by the Portland Cement Association on concrete for residential construction received high honors during the annual convention of the National Association of Home Builders held in Chicago, January 20-22.

Three awards made by a joint committee of the NAHB and the Producers' Council were accepted by S. H. Westby, manager, Housing and Cement Products Bureau of PCA.

BRI conducts 8th annual meeting

Building Research Institute conducted its 8th annual meeting in Pittsburgh, April 7-8.

Six of the nation's largest manufacturers of building products put their research directors on the rostrum to talk about research planned for the 1960's and what it will mean in terms of the design and construction of buildings in the 1970's.

Two technical sessions were devoted to "Sealants for Curtain Walls" covering a variety of subjects such as current status of sealants for curtain walls; performance testing; static tests; dynamic tests; users problems; and new developments in materials and techniques. Proceedings will be published under sponsorship of the cooperating manufacturers.

The technical session devoted to "Mechanical Fasteners in Building" featured a paper on "Fastening or Anchoring to Concrete" presented by Frank A. Werstein, vice-president of the Phillips Drill Co. Proceedings of this session will be published in full by the Industrial Fasteners Institute in its quarterly magazine, *Fasteners*.

Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

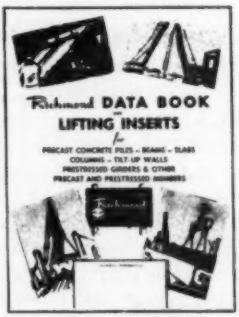
Certified tests by an independent laboratory assure proper strength rating for efficient design and performance in concrete at usable strength

Richmond has developed and tested a complete line of Lifting Inserts and accessories for handling and placing of precast concrete wall slabs, columns, beams, girders, piles, etc.

No matter what type of precast units are involved, Richmond can supply exactly the right type of insert for the specific job.

These units are designed with adequate extra strength and for simple operation in lifting and bracing precast concrete components.

Send for your copy of the Richmond Data Book on Lifting Inserts giving complete technical data, dimensions, working loads and ultimate strengths in various strengths of concrete. At the same time ask for your copy of the latest Richmond Handbook showing the complete line of form tying anchorage and accessory devices for concrete construction backed by 47 years in this field.



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Donald S. MacBride

Donald S. MacBride, president of American Cement Corp., died in March, after a short illness.

Prior to becoming president of American Cement Corp., Mr. MacBride was associated with Hercules Cement Corp. for 20 years, successively as vice-president for sales, executive vice-president, and president. On December 31, 1957, Hercules, Peerless, and Riverside cement companies merged to form American Cement Corp. and Mr. MacBride was elected president and chief executive officer.

Mr. MacBride graduated from the University of Pennsylvania in 1916 with a BS in civil engineering. During World War I he served in the U. S. Army Engineers.

A long-time member of ACI, Mr. MacBride had been associated with the cement industry for the past 40 years, starting his career as a district engineer with the Portland Cement Association. He was a former director of the Portland Cement Association and American Mining Congress, and active in a number of societies.

Alpha Cement announces three executive promotions

Alpha Portland Cement Co. has assigned new and greater responsibilities to two traffic department executives.

James J. Bethune, general traffic manager, retains that title and assumes also the title of assistant to vice-president, sales. Stanley Hawk, eastern division traffic manager, remains in that position and also becomes assistant general traffic manager.

James H. Gourd, salesman for Alpha in the northern New Jersey territory since 1956, becomes eastern district sales manager.

Raymond International honored by Secretary of the Navy

Raymond International Corp. of New York City was among three American general contracting firms honored by the Secretary of the Navy with Certificates of Merit for their part in the $322 million Spanish bases construction program.

The integrated base system in Spain includes three major U. S. Air Force fighter-bomber bases, small material and supply and communications facilities, facilities for storage and transmission of liquid petroleum products, a Naval Air Station with port fuel and ammunition storage facilities, two subsidiary Naval facilities for fuel and ammunition storage, and auxiliary stations.

George F. Ferris, president of Raymond International accepted the award in behalf of the three firms engaged in the joint construction venture.

Bureau of Reclamation seeking engineers

Open examination is announced to fill positions in the Bureau of Reclamation. Employment possibilities from this examination exist in Oregon, Washington, California, Arizona, Nevada, Idaho, Montana, Wyoming, Colorado, New Mexico, Utah, North Dakota, South Dakota, Nebraska, Kansas, Oklahoma, Texas, and Alaska. Appointments will be made to fill vacancies as may exist in presently established project offices or future offices as may be established within these geographical boundaries.

Full details of career opportunities for engineers with the Bureau are covered in Form 57. Apply to Executive Secretary, Central Board of U. S. Civil Service Examiners, Bureau of Reclamation, Denver Federal Center, Denver, Colo.

Errata

The following corrections should be made in "Role of Cement in the Creep of Mortar," by A. M. Neville published in the March, 1959, JOURNAL.

p. 970—in Table 3, column headed "120 days, C_{120}" under "Stress of 2150 psi" for Cement No. 11, change 58 to 48.

p. 975—in Table 7, column headed "28-day cylinder strength, psi" for Cements 2, 2A, and 2B, change 5350, 5400, and 5120 to 3770, 4000, and 3900, respectively.

p. 977—the last sentence of the fifth paragraph from the top of the page should be corrected, in part, to read ". . . usual range of working stresses.[28]"

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
P. O. Box 4754, Redford Station
Detroit 19, Michigan

Date_____

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies	**$15.00**
Individual Members (All other foreign countries)	**12.00**
Corporation Members	**50.00**
Contributing Members	**100.00**
Junior Members—nonvoting (under 28)	**7.50**
Student Members—nonvoting (under 28)	**5.00**

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for _____ **membership**
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by _____

For Corporation Membership, ACI representative will be _____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature _____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency of government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 6. Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
Please Print

Date of Birth _____
Month Day Year

Title or Position _____

Name of Firm or Organization _____

☐ **Business Address** _____

☐ **Resident Address** _____
(Please check address to which you wish mail and publications sent)

Nature of Firm's Business _____

The ACI Membership Directory will be sent—as available—only on request.
Check here if you wish to receive the latest edition. ☐

Honor Roll

January 1—March 31, 1959

Heading the Honor Roll this month is Robert P. Witt with 9½ credits. Samuel Hobbs is second with 7 credits and John E. Heer, Jr. and David A. Sauer are tied in third place with 6 credits each.

Let's expand both ideas and benefits in ACI this year by having "every member propose a member."

Robert P. Witt	9½
Samuel Hobbs	7
John E. Heer, Jr.	6
David A. Sauer	6
Kenneth M. Huber	3
A. T. Klassen	3
Charles C. Luther	3
Rafael Enrique Pacheco	3
C. P. Siess	3
Ferruh Taskin	3
John P. Thompson	3
Henry Aaron	2
Michael Alexander	2
John E. Breen	2
Boris Bresler	2
John L. Burdick	2
S. D. Burks	2
James Chinn	2
W. S. Cottingham	2
Phil M. Ferguson	2
Martin J. Gutzwiller	2
Mauro N. Guzman	2
Robert B. Harris	2
Leo M. Legatski	2
Joachim F. Leppmann	2
Gene M. Nordby	2
Joseph J. Shideler	2
George B. Southworth	2
Glenn C. Thomas	2
A. M. Tovar	2
Lewis H. Tuthill	2
Jose R. Perez Valentin	2
C. K. Wang	2
Roberto Zepeda Aldana	2

New Members

The Board of Direction approved 97 individual applications, 8 Corporation, 16 Junior, and 21 Student applications making a total of 142 new members. Considering losses due to deaths, resignations, retirements, and nonpayment of dues, the total membership on March 1, 1959, was 9833.

Individual

ALDEN, GEORGE C., West Covina, Calif. (Gen. Mgr., Vice-Pres., A & E. Labs.)
ANDERSON, JOHN R., Pasadena, Calif. (Struct. Engr.)
AUBREY, W. H., London, England (Phillips Floors, Ltd.)
BARETSKY, WILLIAM, Utica, N. Y. (Supv. C. E., USAF)
BHARMAL, YOUSUF EBRAHIM, Karachi, Pakistan (Asst. Chf. Engr., Haydari Constr. Co., Ltd.)
BIENIEK, VICTOR J., Springfield, Mass. (Partner, Caolo Assocs.)
BISCHOFF, PAUL, Bay City, Mich. (Vice-Pres., Chem. & Research, Supv. Prod. & Quality Control, Aetna Portland Cement Co.)
BRADFORD, WILLIAM E., Los Angeles, Calif. (Sales Engr., Robert J. Hiller Co.)
BURRES, WARREN G., Los Angeles, Calif. (Distr. Engr. PCA)
CARWARDINE, R. R., Ottawa, Canada (C. E., Est., Job Management, Geo. A. Crain & Sons, Ltd.)
CASHELL, HARRY D., Arlington, Va. (Hwy. Physical Research Engr., Bur. of Pub. Rds., Office of Research)
CHAPMAN, Donald L., St. Paul, Minn. (Head of Struct. & Civil Engrg. Dept., Grover Dimond Assocs.)
CHEN, SHIN, Taipei, Taiwan, China (Chf. Designer, Kung Sin Engrg. Corp.)
CHINN, WILBUR, Berkeley, Calif. (Supv., Oakland Lab., Hales Testing Labs.)
CHUTHARATANA, LT. SEVI, Bangkok, Thailand (Struct. Engr., Tech. Div., The Directorate of Air Installations)

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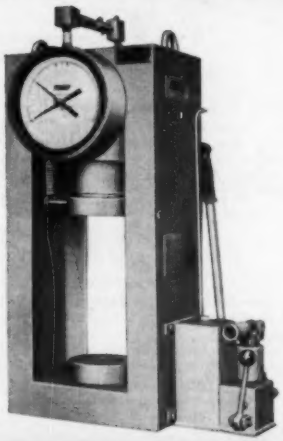

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Engineer: Donald B. Warren Co. and S. B. Barnes
Contractor: Griffith Company
Laboratory: Bureau of Standards, City of Los Angeles
Manufacturer: Griffith Company
Structure: Paving concrete

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WATER-REDUCING ADMIXTURES for CONCRETE

1 IMPROVE CONCRETE QUALITY: —

MARACON® promotes more complete hydration of cement particles and permits a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. Low water content means greater durability and resistance to weathering.

A. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.

B. Decrease permeability.

C. Achieve greater density and higher durability factors.

2 LOWER CONCRETE COSTS: —

A. Maintain slump and workability at low W/C ratios.*

B. Attain higher strengths without increasing cement content of a mix.**

C. Permit economical redesign of conventional concrete mixes.

*Decrease water by as much as 20% . . .
**Increase strengths by as much as 40% . . . depending on the mix design.
(The above figures are not represented as maximum. Greater percentages have been achieved in both respects.)

MARACON also reduces water requirements in concrete mixes containing pozzolanic materials.

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Strain gage recording and plotting system

Strain gage recording and plotting system, developed by B & F Instruments, Inc., plots strain versus load for 24 to 96 strain gage channels on continuous paper loop and is available with a digital readout so that strains can be tabulated on a typewriter, IBM cards, or punched tape.

Each system includes 24 channel modules, the recorder or plotter, a regulated power supply and a programmer. Components are enclosed in one or two 6-ft racks mounted on casters. Complete details available. —B & F Instruments, Inc., 3644 North Lawrence St., Philadelphia 40, Pa.

Nonshrinking grouting material

Emeri-Grout, a nonshrinking grouting material containing Cortland emery, a form of corundum, has been placed on the market by Walter Maguire Co.

Machinery and structural members are seated more securely in Emeri-Grout according to the manufacturer. Company further recommends Emeri-Grout for anchoring and maintaining proper alignment of milling machines, boilers, generators and rectifiers, steam engines, compressors, pumps, ball mills, blowers, cranes, and similar equipment, as well as structural members imposing heavy stresses.—Walter Maguire Co., Inc., 60 East 42nd St., New York City

Silicone water repellent

Water-soluble silicone water repellent to protect concrete surfaces on highways and bridges has been introduced by Silicones Division, Union Carbide Corp. It is called "Union Carbide" XR-20 Silicone. It is shipped as a 20 percent silicone concentrate and can be readily made into a 2 percent ready-to-use solution by mixing 11 parts of water to one gallon of the compound according to the manufacturer. Eight-page illustrated booklet, "Concrete Highway Protection" available on request.—Silicones Division, Union Carbide Corp., 30 East 42nd St., New York 17, N. Y.

Waterstops

Sealtight Hydrojoint PVC Waterstops, extruded from a special compound of polyvinylchloride have been introduced by W. R. Meadows, Inc.

The waterstops are engineered with a unique cross section. The center bulb has been designed to provide the ability to handle pressures caused by concrete movements according to the manufacturer.

Company states that the waterstops are strong, lightweight, and easy to handle; supplied in 50 ft coils, individually packaged. They are flexible and may be bent around corners or formed in curves to meet job requirements and can be spliced on the job.—W. R. Meadows, Inc., 26 Kimball St., Elgin, Ill.

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Plastic-faced concrete block

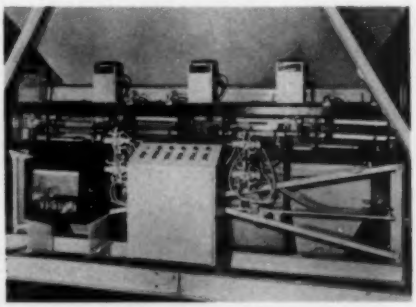
Pre-finished, plastic-faced concrete or lightweight concrete block that permits the erection of finished walls in one operation has recently been introduced. Known as Aristocrat block, this load-bearing construction material eliminates need for lathing, plastering, and painting according to the manufacturer.

The plastic facing is polyester resin combined with marble chips or other decorative aggregate such as oyster shell integrally and permanently bonded to the block by a patented process. Blocks may be used for interior or exterior walls and are available in single or double facing, as well as end facing. Nine standard colors available; special colors may be ordered at slight additional cost. Manufacturer states that Aristocrat block conforms to ASTM Specification C90-44 for Grade A block.—American Cyanamid Co., 30 Rockefeller Plaza, New York 20, N. Y.

Automatic aggregate batcher

Automatic aggregate batcher with four-pen autographic strip chart recorder is available from Blaw-Knox Co. for high speed batch plant operations in highway and airport concrete paving.

The plant's interlocked batchers and recorders meet specifications of the U. S. Army Corps of Engineers for



airport paving according to the manufacturer and has a guaranteed hourly production of 180 accurate 1¼ cu yd batches.

Photograph shows Model P-4150 GW aggregate bin and Model E-41B4 automatic aggregate batcher. The aggregate bin is designed for portability and rapid erection and dismantling.—Blaw-Knox Company, 300 Sixth Ave., Pittsburgh, Pa.

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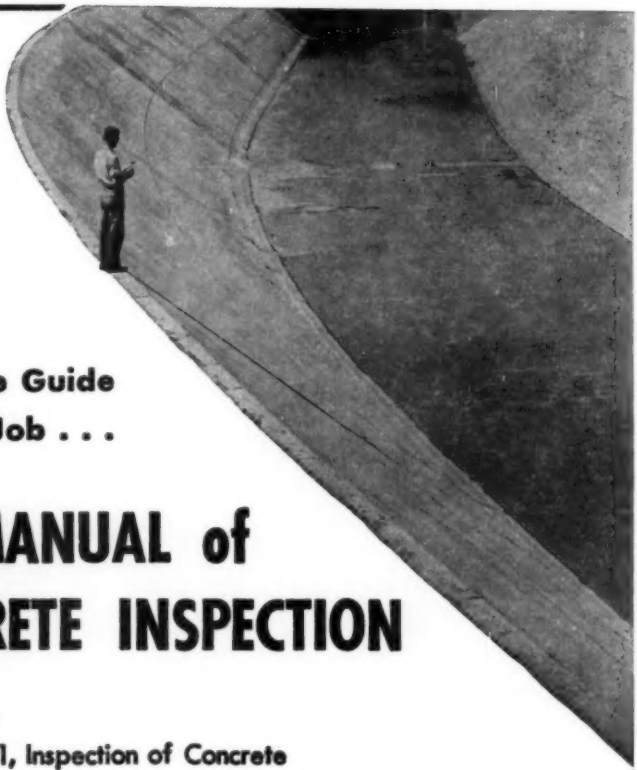

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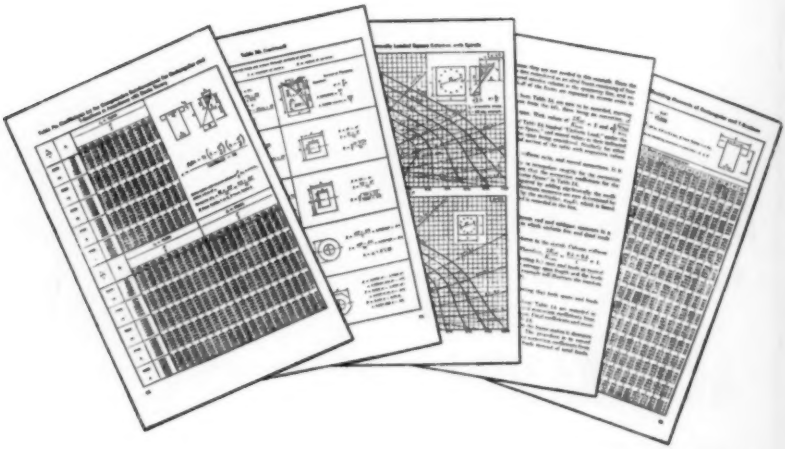

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